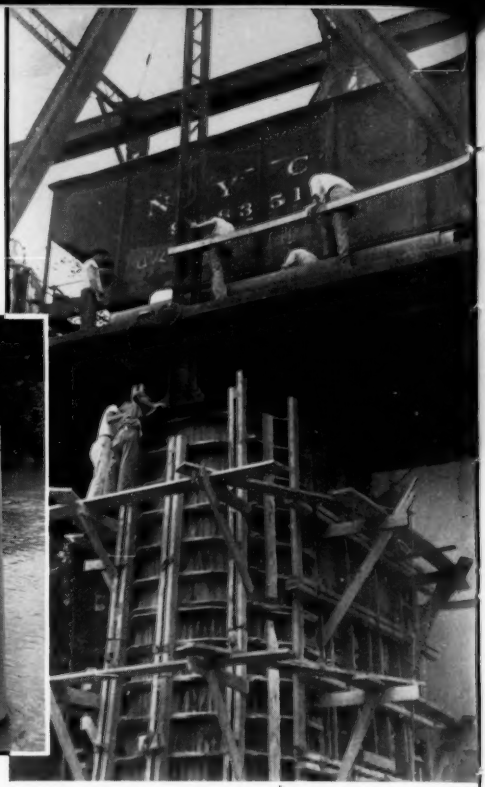
ACI News Letter

Vol. 22 No. 9 JOURNAL of the AMERICAN CONCRETE INSTITUTE May 1951

PICTURE OF EFFICIENCY *Plus*

Under-bridge view showing old pier structure (background), repaired and strengthened by Prepakt, and one of seven new piers constructed with Prepakt materials and methods.



Under-bridge view showing the filling of the new pier forms with coarse aggregate placed directly from a standard railroad hopper car.

No interruption to normal rail traffic!—No stockpiling and material transfer problems! —No excessive plant expenditures!

These all contributed to the efficiency-plus operation in repairing and strengthening the old pier structures and building seven new 35-foot gravity-type piers on this 1212-foot "Monon" Railroad bridge at Delphi, Indiana.

In addition to the "on-the-job" economies made possible with Intrusion-Prepakt methods and materials, the end product is a superior concrete structure. Highly impervious Prepakt concrete provides extremely great resistance to weathering and other deterioration factors.

Full information on Prepakt materials and services as applied to your specific problem may be had by calling or writing the Main Office, Cleveland, Ohio.

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ACI

News Letter Contents

May 1951

On the Cover—A parking garage which utilized unit buildings resembles tables set on top of each other. See p. 669 for a detailed description.

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Research committee reports

The assignment of Committee 115, Research, "is to review and correlate research in concrete and reinforced concrete and to consider research methods and objectives." The committee has attempted to fulfill this assignment through research sessions at annual conventions, through distribution of the annual compilation of research projects under way at various laboratories, and through limited correspondence among committee members.

The annual compilation is divided into three parts. The first part names educational institutions that are conducting concrete research and lists the projects they are working on. The second part provides similar information for noneducational organizations. In the third part the projects are classified under subject headings. This year, 55 organizations in all parts of the country reported to the committee. These organizations included universities, highway departments, federal departments, cement companies, industrial laboratories, commercial laboratories and associations. The reporting organizations listed 346 different projects under way, which cover all phases of research in the fields of aggregates, admixtures, cements, and plain and reinforced concrete. Copies are available from ACI headquarters.

Convention session synopsis

The ten papers presented at the research session of the San Francisco convention covered a large variety of topics.

S. J. Chamberlin, in his report on spacing of reinforcement in beams, presented some preliminary results from single-bar beams with "simulated" variable bar spacing. Indications were that the ultimate load on the beams was about proportional to the "simulated" clear spacing between bars, provided that fracture was not due to bond or tensile failure. Limited load-deflection, load-strain, and load-slip curves were shown.

Continued on p. 4

In a paper prepared by M. L. Mass and presented by J. R. Janney, the results of tests on beams reinforced in the regular manner and prestressed reinforced concrete beams were discussed. The prestressed reinforced concrete beams had 55 percent less steel than the companion beams reinforced in the regular manner, but carried 65 percent greater load at first crack and 43 percent greater load at failure.

Phil M. Ferguson discussed the effect of repeated bond loads on bond-slip curves of bars already carrying axial tension. He pointed out that, while the tests were only exploratory in nature, initial axial tension in the bar increased the slip and seemed to reduce the ultimate bond strength. He presented bond-slip curves which showed the accumulative effect of unloading and reloading cycles both with and without initial axial tension.

Stanton Walker, in his talk on performance of automatic freezing and thawing equipment for testing concrete discussed the construction, operation and performance of his equipment, which permits 8 cycles of freezing in air to 0 F and thawing in water at 40 F.

Thomas B. Kennedy reported on an investigation of a pressure method for determining the air content of hardened concrete. His discussion showed that the voids in hardened concrete respond to variations in outside air pressure as do voids in plastic concrete, and consequently can be calculated by a formula based on Boyle's law. The apparatus used for this determination is similar to the Washington State air meter. Mr. Kennedy indicated that the method was satisfactory but that occasionally a relatively long period was required to bring the hardened 10 x 20-in. cylindrical specimens to an equilibrium condition; in some cases, as much as 116 hours.

T. C. Powers, in speaking of resistance of concrete to freezing and thawing, described his conception of the mechanism of frost action in hardened portland cement pastes. He presented experimental evidence that the destructive effect of freezing is due to pressure in the unfrozen water and not to what might be called crystal pressure. He showed that if water has access to voids, the only pressure developed during freezing will be that caused by the resistance to flow of the unfrozen water through the submicroscopic pores of the paste to the voids. Such pressure must be zero at a void boundary and must increase with distance from the boundary, the pressure for a definite distance inward being below the bursting strength of the paste. Thus, each void (air bubble) in the paste is bordered by a shell of paste protected from disruptive pressures. When the voids are so numerous that they are not more than about 1/100 in. apart, the protected shells overlap sufficiently to prevent disruptive pressures throughout the paste. The experimental program of the Portland Cement Assn. laboratories designed to test this hypothesis was described.

Bailey Tremper discussed electrical heating of concrete pavements for ice prevention. He pointed out that his experiments were aimed at determining the heat required to prevent adherence of ice to pavements and not necessarily the heat required to melt heavy falls of snow. His work involved the evaluation of factors needed for design purposes, and from his experiments he indicated that to maintain the surface of a dry slab at 40 F when the atmospheric temperature is 0 F requires from 1800 to 3000 kilowatts per mile of 24-ft pavement.

H. S. Meissner presented a report on effect of thermal differences in aggregate on concrete deterioration. He noted that factors other than large differences in the thermal coefficients of aggregate and mortar may be responsible for poor durability of concrete. He called attention to the anisotropic characteristics of several aggregates, and also pointed out that deposition of calcium hydroxide on the cleavage planes of certain aggregates due to wetting and drying action of concrete may be important.

C. H. Scholer spoke on a new and unusual topic, the use of radioactive salts in studying certain properties of concrete. He described the techniques and procedures that had to be developed in using these salts, and then described a study of the movement and distribution of radioactive alkali salts in concrete.

David Watstein spoke on the effect of rate loading on the compressive strength and modulus of elasticity of concrete. He discussed the effect of straining rates as high as 1,000,000 times those normally used in

determining the compressive strength and modulus of elasticity of concrete. In his conclusions, he pointed out that high loading rates increased the compressive strength to 1.8 times the static strength for weak concrete, and to 1.4 times the static strength for strong concrete. The secant modulus of elasticity of weak concrete under high loading rates was about 1.5 times as large as the modulus determined statically, and for strong concrete the secant modulus was about 1.2 times larger than the static value.

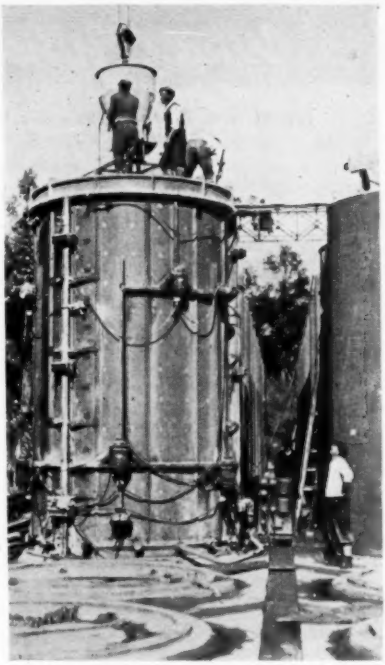
From the continuing interest of ACI membership in the activities of Committee 115, it is evident that real needs are being satisfied. It would be presumptuous, however, to assume that the committee has reached the zenith of its possible usefulness; there must be various ways in which increased service can be rendered. Your reactions and suggestions in this connection are earnestly requested. Since the committee is anxious to have complete coverage of all research projects in its next compilation, all persons having suggestions or knowing of omissions are urged to initiate correspondence at once. No information will be given any publicity without the permission of the correspondent. Address your suggestions and inquiries to:

Prof. George W. Washa
Secretary ACI Committee 115
Engineering Building
University of Wisconsin
Madison 6, Wis.

Bar cards available

The Concrete Reinforcing Steel Institute has supplied ACI headquarters with "bar cards" for A 305 reinforcing bars for distribution to Members for the asking. The card lists both old and new bar size designations, weight and nominal dimensions (diameter, cross sectional area, perimeter).

Members may receive a copy of the card by writing ACI headquarters.



Vacuum concrete pipe cast in Italy

Large concrete pipe cast at Mucone, Italy, use the vacuum process to advantage. The pipes are 16 ft long and have an inside diameter of 10 ft. Inside and outside forms are removed 30 minutes after casting. The same forms can be used 6 to 8 times each day.

Public Health Assn. favors performance codes

The committee on hygiene of housing of the American Public Health Assn. in a report, "Construction and Equipment of the Home," takes exception to specification-type codes in force in American municipalities.

The report declares that "the only satisfactory solution to this problem of obsolete codes lies in the replacement of outmoded specification codes by performance codes which set forth the basic objectives to be attained, and permit the use of any materials or methods that will adequately attain the ends in view."

Who's Who This Month

L. G. Farrant and W. C. Harry

"Unit Buildings Cut Construction Costs," p. 669, introduces L. G. Farrant and W. C. Harry, consulting engineer and structural designer, respectively, of the L. G. Farrant consulting firm, Miami.

Mr. Farrant, born in Australia, studied in England and worked with consulting engineers, architects and contractors there for 10 years. He entered private practice as a consulting engineer when he came to the United States four years ago.

He has specialized in flat plate structures and precast concrete framework. Besides the New Orleans parking garage, other recent flat plate projects have included an office building in Nassau, Bahamas; garage, Dallas, Texas; apartment building, Miami; store, Atlanta, Ga.; and warehouse in Richmond, Fla. He is also a lecturer at the University of Miami.

Mr. Harry, a graduate of Cornell University, besides working as structural designer is also a lecturer at the University of Miami.

Committee 312

Supplementing two previous reports in 1932 and 1940, Committee 312 submits the third of a series, "Plain and Reinforced Concrete Arches," on p. 681.

Charles S. Whitney, chairman, and Boyd G. Anderson co-authored the report for the committee. Both are associated with the consulting firm of Ammann and Whitney, New York.

The committee also includes W. S. Cottingham, University of Wisconsin, Madison, and Clyde T. Morris, Ohio State University, Columbus.

W. J. McCoy and A. G. Caldwell

A "New Approach to Inhibiting Alkali-Aggregate Expansion," p. 693, is suggested by W. J. McCoy and A. G. Caldwell, director of research and research chemical engineer, respectively, Lehigh Portland Cement Co., Coplay, Pa.

Mr. McCoy graduated from Wisconsin State College and received a master's degree from the University of Minnesota. He has been with Lehigh Portland Cement Co. since 1938. A member of Committee 115, Research, he is also active on ASTM committees on cement, concrete and mortars.

Mr. Caldwell has been with Lehigh Portland Cement since 1934. He graduated and received an MS from the University of Illinois. He devotes considerable time to studies of lightweight aggregates and volume change characteristics of mortar and concrete.

Committee 805

Committee 805, under the chairmanship of W. L. Chadwick, vice-president in charge of engineering and construction, Southern California Edison Co., Los Angeles, proposed the new standard on shotcreting in the November 1950 JOURNAL. "Recommended Practice for the Application of Mortar by Pneumatic Pressure," p. 709, was adopted at the San Francisco convention and ratified by ACI membership in April.

The committee includes J. A. McCrory, Shawinigan Engineering Co., Ltd., Montreal, Que., Canada, and R. B. Young, Hydro-Electric Power Commission of Ontario, Toronto, Ont., Canada.

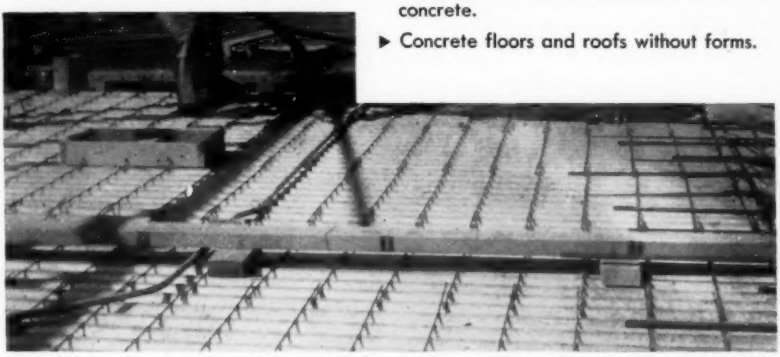
Committee 617

ACI Committee 617, under the chairmanship of H. F. Clemmer, materials engineer, Government of the District of Columbia, proposed revisions of the pavement standard in the October 1950 JOURNAL. The revised standard, "Specifications for Concrete Pavements and Bases (ACI 617-51)," p. 721, was adopted at the 47th annual convention in San Francisco and ratified by letter ballot in April.

Other members of the committee are A. T. Goldbeck, National Crushed Stone Assn.; W. E. Hawkins, North Carolina State Highway Dept; Fred Hubbard, National Slag Assn.; Frank H. Jackson, Bureau of Public Roads; Ted J. Kauer, Ohio Dept. of Highways; Bailey Tremper, Washington State Dept. of Highways; Stanton Walker, National Ready Mixed Concrete Assn.; and M. Allan Wilson, Delaware State Highway Dept.

COFAR

- ▶ Reinforced concrete without forms.
- ▶ High-strength, deep-corrugated steel manufactured with welded, closely spaced transverse wires (T-wires).
- ▶ Positive reinforcement permanently anchored to and combined with structural concrete.
- ▶ Concrete floors and roofs without forms.



STRONG

Guaranteed yield point (.1% offset) over 80,000 psi effective as reinforcement, balanced design, up to full tensile strength over 100,000 psi, independent of form stress.

VERSATILE

Minimum 24 ga. —(A_s .35 sq. in. per foot width); maximum 18 ga. —(A_s .73 sq. in. per foot width); to meet full range concrete slab design. COFAR reinforcement satisfies all normal concrete slab design.

Cold-drawn, high-strength T-wires not over 6″ c.c. are welded to the deep-corrugated steel, in manufacture, and constitute temperature reinforcement in the slab, mechanical anchorage and positive shear transfer from concrete to steel.

PERMANENT

COFAR sheets hot-dip galvanized, with specification heavy galvanizing, provide complete permanence for normal interior exposures. Concrete and steel permanently bonded by chemical union of calcium zincate for all normal loads. Fire resistance for any exposure by lightweight, modern ceiling plaster or fibre protection.

ECONOMICAL

COFAR ***is concrete reinforcement,*** steel weight substantially equal to conventional reinforcement; COFAR ***also is the concrete form.*** Form sheeting as well as concrete runways are eliminated. COFAR comes detailed and cut to fit each job. Important material, labor, and time savings are made.

Advice for design engineers, review and estimates. COFAR is available — Granco Steel Products Company.

GRANCO STEEL PRODUCTS CO.
(*Subsidiary of* GRANITE CITY STEEL CO.)
GRANITE CITY, ILLINOIS

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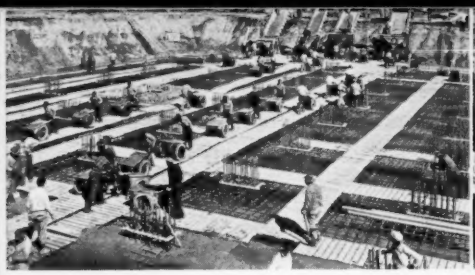


1. Feb. 27, noon

2. Feb. 27, noon

"Plan your work—then w

2800 cu yd of concrete placed in 30 hours

Probably about as straightforward a major building operation as could be found is the Calvert Distilling Co. warehouse project at Relay, Md., being constructed by Baltimore Contractors, Inc. However, a recent phase of construction points out the importance of job planning, especially for big "concrete pours," in this case a 170 x 130 ft slab.

After excavation, the earth was graded and covered with heavy moisture resistant paper. Foundation wall, stair well and elevator pit forms, and floor drains, had been built; about 165 tons of reinforcing steel were in place. The reinforcing was concentrated at 70 locations where 40 interior columns and 30 exterior columns would rise, and in two horizontal planes, one near the bottom and one near the top of the slab. Chutes, runways, access ladders and concrete carrier buggies were in place and ready for concreting the 42-in. slab.

An important part of preparations, which was not used, was the large steam boiler and tarpaulins to cover the working area. Had the weather turned cold enough to freeze earth and concrete the whole area could have been steam heated under canvas.

With allowances for departure from strict anticipation of materials needed, transportation time and manpower coordination, plans were completed on February 26 for the continuous pour.

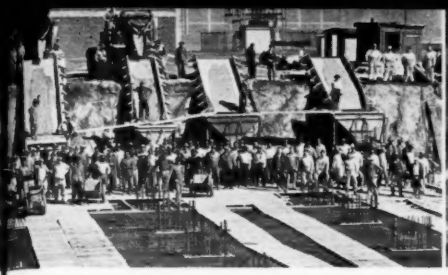
The ready-mix plant allocated 40 transit-mix trucks and it was planned to deliver 100 cu yd of 3000-lb concrete per hour into the chutes. It was estimated that 2800 to 2900 cu yd would do the job and that continuous placing would take about 30 hours.

Plans called for 40 hand pushed, rubber-tired buggies and a crew of about 100 men continuously on the job, *i.e.*, two crews and three shifts, starting at 7:30 a.m. and 6:00 p.m. February 27, and 7:30 a.m. February 28. Actually the supervisory staff worked almost throughout the entire operation.

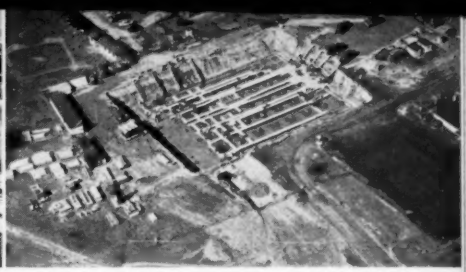
Fig. 1 is at noon Tuesday, February 27. The weather was favorable and temperatures above freezing. Sometimes, four trucks would be unloading into distribution hoppers at one time and one truck would be chuting concrete via a smaller chute directly into the slab along the boundary wall. There were a few rare moments when no truck was unloading and 40 buggies and pushers would be idle.

Fig. 2 is also at noon Tuesday. The placing started farthest from the truck unloading point. The concrete was vibrated carefully, with special emphasis at all border areas, adjacent to forms, top and bottom surfaces close to major horizontal reinforcing, space within reinforcing for vertical columns, and the junction of buggy loads.

At the end of 5 hours, 504 cu yd of concrete were in place. As the proper depth was reached the sectional runways were lifted and placed in new locations. The cement finishers followed closely, taking particular care to step always on the top level reinforcing rods; to slip off usually required assistance in withdrawing a concrete encased foot and leg.



3. Feb. 27, 12:30 pm



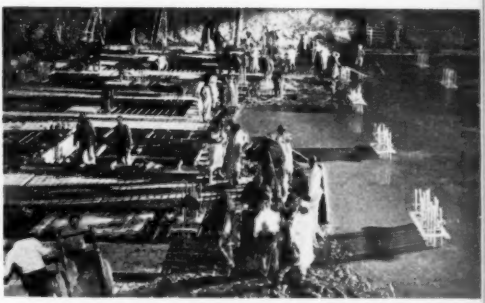
4. Feb. 27, 1:30 pm

en work your plan"

Fig. 3 is about 12:30 p.m. and seemingly shows most of the crew taking time off for lunch. That is not the case; only two minutes were allowed for the picture. Sandwiches, coffee and soft drinks were provided to supplement lunches which the men brought, and only a small group at a time took time out to eat. Including eating time and change of shifts, 10 cu yd was placed every 6 minutes. There was not a moment's let-up.

Fig. 4 is an aerial view taken at 1:30 p.m. The new structure will be similar to the existing warehouses. The photo shows an over-all view of the excavation; transit-mix trucks coming, unloading and returning empty; men placing and finishing concrete; and that portion of the bottom slab finished.

Fig. 5 is at 8:00 p.m. Tuesday night. Powerful floodlights enabled the men to work as effectively at night as in daylight.



5. Feb. 27, 8:00 pm

6. Feb. 28, noon

Then came the dawn and a fresh shift at 7:30 a.m. Wednesday, February 28. Hour by hour around the clock the schedule clicked with precision.

Fig. 6 is at noon Wednesday, 28 hours and 30 minutes after starting. There was room left for relatively few men to place concrete. The rest had been assigned to arrange the no longer needed walkways and to clean-up operations. The pour was completed at the end of 30 hours with no important accident or incident. 2817 cu yd of concrete had gone into the slab in 600 truck deliveries.

So after all there was nothing unusual—except perfect planning, coordination and cooperation—which produced the slab on time (with some thanks for the weather).

The project is a good example of the builder's rule, "Plan your work, then work your plan."

ACI receives ECA Certificate of Cooperation

A Certificate of Cooperation was presented in April by Mayor Albert E. Cobo of Detroit to the American Concrete Institute on behalf of the Economic Cooperation Administration.

The certificate was given in recognition of ACI's assistance and cooperation in helping technical missions from overseas studying United States construction planning, equipment and building techniques, and recommending various plant visits and conferences for these missions so they could get first-hand knowledge of American construction methods.

THE ECONOMIC COOPERATION ADMINISTRATION

CERTIFICATE OF COOPERATION

awarded to

American Concrete Institute

for furnishing technical assistance to the Peoples of the Marshall Plan Countries to aid them in maintaining individual liberty, free institutions and peace.

April 3, 1951

Paul G. Hoffman
ADMINISTRATOR

The Technical Assistance Program was begun by ECA to help Western Europe increase production by better utilization of existing plant and equipment, through the study of effective utilization of plant, equipment and manpower in the U.S.

Positions and Projects — ACI Members

ACI Members elected to NSPE posts

John F. Reynolds and Russell B. Allen were among those recently elected to office by the National Society of Professional Engineers.

Mr. Reynolds, of Reynolds, Smith and Hills, Jacksonville, Fla., was elected vice-president of the Southeastern Area. A graduate of Iowa State College, he is also a member of Tau Beta Pi, ASCE and the Society of American Military Engineers.

Professor Allen, Maryland University, College Park, Md., was reelected to a fourth term as treasurer of the society. Graduated from Yale University, he is president of the executive committee of the Maryland Chapter of the American Assn. of University Professors, has been national vice-president of Tau Beta Pi, and affiliated with the Society for Promotion of Engineering Education and Sigma Xi.

Ateshoglou opens office

A. D. Ateshoglou, formerly with J. Di Stasio and Co., has opened a new consulting engineering office, Hoffberg and Ateshoglou, in New York City.

Nichols named "engineer of year"

Marvin C. Nichols, associated with Freese, Nichols and Turner, Fort Worth, Texas, was honored recently as "engineer of the year" by the Texas Society of Professional Engineers of that city. Mr. Nichols was cited for his "untiring efforts on behalf of the engineering profession and unselfish service to this society and his fellow engineers . . ."

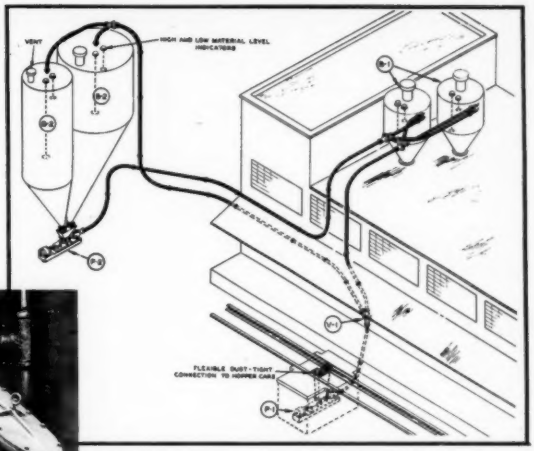
Mass joins research foundation

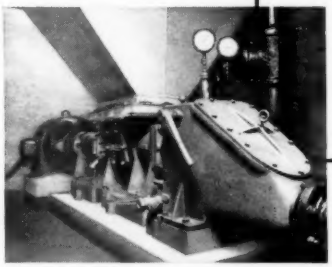
Marvin L. Mass, formerly assistant professor of civil and architectural engineering, University of Colorado, has accepted a position as research engineer with the Armour Research Foundation of Illinois Institute of Technology, Chicago.

Hamlin and Kinnerman have new duties

Horace P. Hamlin, designing engineer, Raymond Concrete Pile Co., New York, has been appointed consulting civil engineer. Mr. Hamlin has been connected with the organization for 43 years. William P. Kinnerman, who has been assistant civil engineer for the firm, has assumed new duties as chief civil engineer.

IF IT'S PULVERIZED . . . A FULLER-KINYON SYSTEM CAN PIPE IT!





F-K System in a cement products plant handling cement and fly ash. Conveys from cars (P-1) to either process bins (B-1) or storage (B-2,) by use of valve (V-1) in the conveying line. Conveys (P-2) from storage to process.

Safe • Simple • Clean • Economical

For these reasons the Fuller-Kinyon System, pumping aerated, pulverized materials, is the *best possible method of transporting such materials!*

Safe? A Fuller-Kinyon System has no hazardous moving parts. Absolute safety from explosion is assured. *Simple?* Flexibility of layout allows conveying pipe lines to be placed overhead, on simple hangers, or underground, to bins in any part of your plant. No bridges or monitors are needed, no changes in building layout or equipment. *Clean?* Material is always completely enclosed. There is no dust nuisance. *Economical?* Power consumption is low per ton of material handled. Maintenance costs are rock-bottom.

If you want to cut down your material handling costs, get in touch with a Fuller engineer. He'll study your layout and recommend the system that will best increase your handling efficiency. There is no obligation.

Typical materials handled:

cement • chalk • coke dust • dolomite • flue dust • fly ash • fuller's earth • graphite • gypsum • kaolin • lime • pulverized coal • rock dust

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Chicago 3 • 120 S. LaSalle St.
San Francisco 4 • 420 Chancery Bldg.

Fuller

P-127

Steinman awarded medal

David B. Steinman, well-known bridge designer, New York, was recently selected by the Board of the Columbia Engineering School Alumni Assn. to receive the Egleston Medal for 1951. This is the highest honor conferred by the alumni association and was awarded in recognition of outstanding professional achievement.

Dr. Steinman was also recently elected to membership in Sigma Xi, national honorary scientific society.

Mather elected C-9 secretary

Bryant Mather, chief of the Special Investigations Branch, Concrete Research Division, Waterways Experiment Station, Corps of Engineers, Jackson, Miss., has been elected secretary of ASTM Committee C-9 on Concrete and Concrete Aggregates. He succeeds Stanton Walker, director of engineering, National Sand and Gravel Assn. and National Ready Mixed Concrete Assn., who had held the position 1926-32 and 1942-51.

Mr. Mather, named to ACI's Technical Activities Committee at the San Francisco convention, is the sixth secretary of Committee C-9 since its organization in 1914. He has been a member of the committee since 1946 and is chairman of the subcommittee on methods of testing and specifications for admixtures for concrete.

Manning elected company president

Vice-Admiral John J. Manning (CEC), USN, Ret., has been elected president and director of James Stewart and Co., Inc. He will be in charge of all operations of the 107-year old engineering and construction organization which was recently purchased by M. Seth Horne and Associates.

Admiral Manning was formerly Chief of Civil Engineers and Chief of the Bureau of Yards and Docks, Department of the Navy.

Originally organized in 1844, the company has built private and public structures throughout the world. The company pioneered in the construction of grain elevators, as well as other structures, and were early developers of sliding forms. They designed the first submarine nets for the British navy in World War I and were building railroads in the West when bison still roamed the plains. One of the first modern textile mills in the South is among their accomplishments. More recent construction includes the naval base in Trinidad, British West Indies, railroad terminals, steel and paper mills, industrial plants and grain elevators.

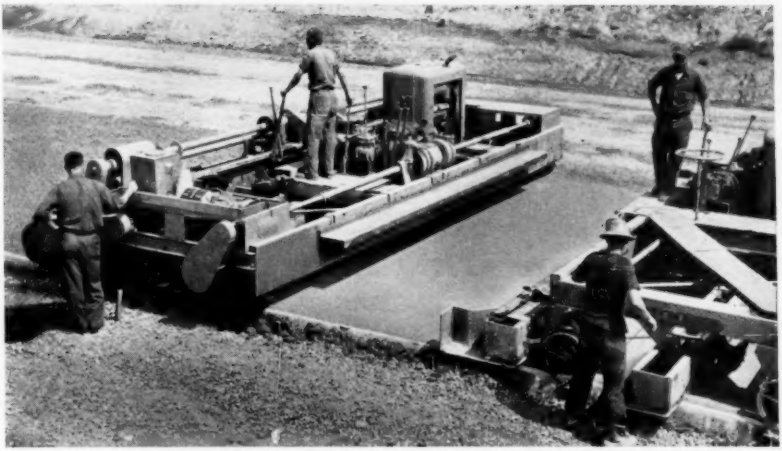
Collins conducts construction seminar

An eight-week course in precast reinforced concrete tilt-up construction, conducted by F. Thomas Collins, consulting engineer, San Gabriel, Calif., had about 40 West Coast contractors enrolled. The seminar, meeting one night per week, reviewed precast practice and emphasized present practice, including ribbed slab technique, flat slab technique and use of erection equipment. Mr. Collins also explained the patent situation.

In describing equipment, he told how the contractor can get by with the least possible capital investment.

The erection portion of the course covered rigging and compared vacuum lifting and point pickup of precast slabs. Forming techniques were described; reinforcing and joinery were studied and the contractors heard how to combine precast with cast-in-place concrete to advantage.

Cost of precasting and estimating received attention, as well as methods of construction and relation of building regulations to precast construction.



Precision Slab -- Faster and at Lower Cost --with Jaeger 3-Screed Paving Team

Machine #1—Jaeger Screw-Screed Spreader that remixes and compacts, spreads, strikes off to approximate grade, *then makes precision-metering strikeoff with its oscillating screed.* Machine #2—Jaeger Type "X" Finisher with transverse front screed and diagonally adjustable rear screed.

It's a 3-screed paving team that saves costly hand labor and costlier paver delays, has also eliminated need for a second finisher on a number of high speed paving jobs. Photo at top shows Screw-Screed Spreader ahead of Finisher. Note semi-finished, exactly-metered surface. Photo at left shows how spreader metering action assures just right roll of material ahead of finisher screeds—no excesses, no deficiencies, uniform compaction.

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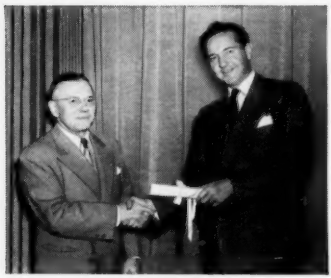


Carl A. Menzel (left) receives the 1951 Award of the Concrete Reinforcing Steel Institute from Henry C. Turner, Jr., president of the Turner Construction Co. and chairman of the award committee.

Menzel wins CRSI award for development of A305 bar

Carl A. Menzel, manager, Housing and Cement Products Bureau, Portland Cement Assn., Chicago, was given the 1951 Award of the Concrete Reinforcing Steel Institute at its 27th annual meeting "in recognition of noteworthy contributions to the advancement of reinforced concrete construction."

Mr. Menzel received the award for his research and development work on the new-type deformed reinforcing bar, which has substantially greater bonding strength than plain bars or conventional deformed bars. The new bar makes possible further economies in reinforced concrete design by providing sounder, more permanent structures, with less steel and at lower cost. Mr. Menzel's work resulted in ASTM Specification A305—the first recognized standard on reinforcing bar deformations. ACI recently revised its Building Code to take advantage of the new A305 bar. Deformed bars complying with Specification A305 are now being rolled by every large producer of reinforcing steel in the United States.

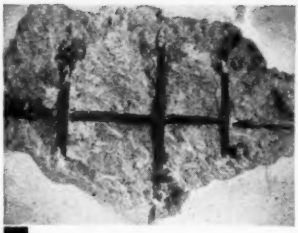
Mr. Menzel's research leading to development of the A305 bar began in 1935. As far back as 1902 various researchers had studied bonding characteristics of reinforcing bars. These men, including Hatt, Talbot, Withey, Abrams, Slater, Richart, Gilkey, Wernisch, Mylrea, and Davis, contributed a large and useful knowledge on the subject. However, it remained for Mr. Menzel to develop a standard specification for a bar that could be rolled economically, yet having bonding characteristics lacking in conventional bars.

Mr. Menzel is well known for his extensive research and development work in many phases of concrete construction. In 1932 he received the Dudley Medal of ASTM for his investigation of the fire resistance and stability of concrete masonry walls. ACI, of which he has been a member since 1935, honored him with the Wason Medal for the "most meritorious" paper of the year for his 1947 paper "Development and Study of Apparatus and Methods for the Determination of the Air Content of Fresh Concrete." He is also the author of many other authoritative papers on concrete and concrete products, and is the designer of an autoclave comparator and molds—now universally used to measure soundness of cement.

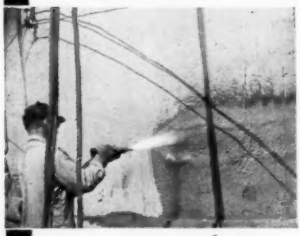
A graduate of the University of Illinois, Mr. Menzel received his bachelor's degree in mechanical engineering in 1917. He began his career in machine shop work—acquiring practical experience which he considers invaluable as a background for his present investigative work. Since 1918, he has been engaged almost continuously in research, testing and development work on building materials and construction. Before joining the Portland Cement Assn. in 1928, he was associated with the Forest Products Laboratory, Madison, Wis., and with Underwriters' Laboratories, Chicago.

At the Portland Cement Assn., Mr. Menzel has served in the laboratories until recently, first as associate engineer, and then as senior research engineer. He was appointed to his present position as manager of the Housing and Cement Products Bureau in 1947.

The Award Committee of the Concrete Reinforcing Steel Institute is composed of one representative from each of three national organizations: Henry C. Turner, Jr., committee chairman, president of the Turner Construction Co., New York, and representing the Associated General Contractors of America; Charles S. Whitney, of Ammann and Whitney, consulting engineers, New York, representing the American Concrete Institute; H. J. Gilkey, head of Department of Theoretical and Applied Mechanics, Iowa State College, Ames, representing the American Society of Civil Engineers.



Typical example of spalling. Note corrosion of reinforcing rods exposed by disintegrating concrete.



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Frederick E. Turneaure

Frederick E. Turneaure, dean emeritus of the University of Wisconsin's college of engineering, died recently. Before his retirement in 1937, he had headed the engineering college for 33 years. He was an authority on the theory of structures, theory and practice of reinforced concrete and in the field of water supply.

He was elected an Honorary Member of ACI in 1932 "in recognition of his service through research and writings for the advancement of the knowledge and use of reinforced concrete." He was also recipient of the Institute's Henry C. Turner Medal in 1930 for "distinguished service in formulating sound principles of reinforced concrete design."

A graduate of Cornell University in 1889, he studied abroad, 1895-96, and received a doctor's degree from the University of Illinois in 1905.

After serving as instructor in engineering at Washington University, St. Louis, 1890-92, Dean Turneaure joined the University of Wisconsin staff as professor of bridge and sanitary engineering. In 1902 he was named acting dean of the college of engineering; made dean in 1904.

He was active in various technical societies and author of numerous books and engineering articles.

Theodore T. Knappen

Theodore T. Knappen, internationally known civil and hydraulic engineer, New York, died recently. Senior partner in the firm of Knappen, Tippetts, Abbett Engineering Co. since 1942, Mr. Knappen had handled numerous foreign engineering assignments. He was a 1920 graduate of the U. S. Military Academy at West Point and served as a lieutenant in the Army Corps of Engineers. In 1928, after work with private engineering firms, he rejoined the Corps of Engineers in a civilian capacity, specializing in flood control work. From 1937 to 1942 he was with the firm of Parsons, Klapp, Brinckerhoff and Douglas, New York consultants, organizing and managing the firm's South American office in Caracas, Venezuela. He was author of many papers on soil mechanics and hydraulics.

Wayne University host to concrete conference

Builders, engineers, architects and products manufacturers gathered in Detroit, April 25 and 26, for a concrete conference sponsored by the Department of Civil Engineering, Wayne University, and Portland Cement Assn. Modular design, concrete masonry, precast and prestressed concrete were featured in discussion.

Modular design

With rising construction costs, ways and means must be found to reduce costs, said John C. Thornton, architect, Detroit Edison Co. He offered modular design as an important factor in reducing costs.

Advantages of modular coordination include simplification of planning and saving of drafting time; easier substitution of alternative materials and changes in specifications; simplification of cost estimating; easier and more economical erection, with a reduction in cutting, waste and fitting on the job-site; and reduction of number of stock sizes of materials and components.

Mr. Thornton said that factors influencing the adoption of a 4-in. module were that it was an even division of a foot, it caused little change in present products and that if units were changed to metric system, there would be small modification.

The concrete products industry has made great strides in manufacturing units to agree with modular design, he said. It cuts material waste and once all phases of the building industry adopt modular coordination it will offer a more effective method of construction.

At the noon luncheon at the Engineering Society of Detroit, David D. Henry, president, Wayne University, spoke on "Education and the National Defense."

Concrete masonry construction

"The moisture content requirement for concrete block is just as important as strength and absorption, but is not checked as often," said S. H. Westby, assistant manager, Housing and Cement Products Bureau, Portland Cement Assn., Chicago. Shrinkage stresses will be reduced if moisture content is minimum.

He listed factors to consider in concrete masonry construction. Blocks stockpiled on the job should be protected from rain and snow; also it is wise to cover the top of an unfinished wall with some type of cover to prevent rain from getting into cells. He recommended that the block should not be prewet before mortar application.

He emphasized that the architect and builder should anticipate cracks with control (dummy) joints. They should usually be 20 to 35 ft apart in concrete masonry construction. The dummy joints should be sealed with calking compound. If the wall is to be painted with cement paint, a coat of shellac over the calking compound will prevent the oil from coming through to discolor the paint.

He also emphasized the desirability of modular coordination. In noting the use of concrete in wall construction, Mr. Westby said that "two-thirds of all masonry walls are concrete block."

William E. Kapp, Detroit architect, said that keeping water out of a structure was a serious problem. He described a 300-ft curved block wall that had no shrinkage cracks. This was possible by completely drying the block. On delivery to the job site, the blocks were protected and also laid on their side so that wind could blow through the openings to aid drying.

Edward Bergman, Cunningham-Limp Co., Detroit, emphasized that lateral stiffness must be provided in control joints.

Precast floors

F. N. Menefee, professor of engineering mechanics, University of Michigan, Ann Arbor, concluded the first day's program with a description of recent developments in precast reinforced concrete floor systems. He discussed the Dox floor and roof system, the F and A system, Joistile, Strestcrete and the Farhan unit.

Precast structural elements

Edward B. Oberly, construction consultant, Portland Cement Assn., New York, opened the Thursday program with an illustrated discussion of precast structural elements.

"All types of construction have been faced with high costs," he said, "so standardization is coming to the forefront." He discussed various projects that used precast units to advantage.

This Johnson Wax Research Tower, Racine, Wisconsin, designed by Frank Lloyd Wright, is outstanding in its many engineering and construction accomplishments. One of the features "behind the scenes" was the 5,000 psi concrete which had to be produced with $\frac{3}{8}$ inch gravel and 6 inch slump because of close spacing of reinforcing steel. Master Builders Pozzolith enabled the builders to meet these extreme conditions. In 1950 alone, over 8,000,000 cubic yards of concrete were produced with Pozzolith — the cement dispersing, water reducing admixture.

He noted that an economical way of casting slabs was to place them on top of each other, thereby needing only edge forms. The slabs can be kept from bonding to each other by using paper or lacquer coated with an oil film.

In jointing precast slabs he cited one job that used sponge rubber as a water seal for keyed panels. Another project, Onondaga County War Memorial, Syracuse, N. Y., used 8 x 8 ft precast panels, 3½ in. thick, as exterior forms for an interior monolithic wall. He also mentioned a powerhouse project in Ohio that used precast sandwich walls, with one panel erected every 3 minutes.

Wai-Jum Yee, of Gruen and Krummick, Detroit, described the design of precast elements for a suburban shopping center where columns, girders, beams and floor slabs would be precast.

"The Building of Detroit" was the topic of after-dinner speaker George R. Thompson, city engineer, Detroit, at the Thursday luncheon.

Prestressed concrete

"The applications are unlimited when prestressing is combined with precast elements," declared Paul F. Rice, district structural engineer, Portland Cement Assn., Detroit. Economy of material, especially for spans of 40 ft or more, was cited as one advantage of prestressing. He also noted that prestressed concrete often offers a superior material when compared to conventional materials.

He reviewed the various prestressing systems and different types of anchorage, and discussed briefly the theory and application.

Technical organizations cooperating in the conference included ACI, Michigan Section of ASCE, AGC, Builders Assn. of Metropolitan Detroit, Concrete Products Assn. of Detroit, Michigan Society of Architects, and Michigan Society of Professional Engineers.

ASTM considers freezing and thawing tests

The formulation of standard freezing and thawing tests on concrete was discussed by Committee C-1 on Cement and Committee C-9 on Concrete and Concrete Aggregate during ASTM Committee Week in Cincinnati recently. Committee C-9 will be responsible for the development of the standard.

Committee C-9 will also formulate a standard chemical test for alkali-reactivity of aggregates, using as a basis the Bureau of Reclamation's "quick chemical test" and the method used by the Corps of Engineers. Another proposed method in this field is the mortar bar test (ASTM designation C 226 T). Two other test methods under development involve potential volume change and abnormal expansion of cement-aggregate combinations.

NSPE to meet in Minneapolis

The National Society of Professional Engineers will hold their 16th annual meeting June 13-16 in Minneapolis.

ASTM meeting to feature symposium on flame photometry

Seven technical symposiums and an estimated 450 meetings of various technical committees are on the program of the 54th annual meeting of the American Society for Testing Materials, Atlantic City, June 18-22.

A symposium on flame photometry will feature a paper on the use of Perkin-Elmer and Beckman instruments for determination of alkalies in portland cement by J. J. Diamond and Leonard Bean, National Bureau of Standards.

Applications of flame photometry to materials investigations will be discussed by J. L. Gilliland, Bureau of Reclamation; W. J. McCoy and G. G. Christiansen, Lehigh Portland Cement Co., will describe the use of the flame photometer for determining lithium in portland cement.

Other symposiums include those on structural sandwich construction, acoustical materials, consolidation testing of soils, and surface and subsurface reconnaissance.

New York surveys municipal building codes

A survey made by the New York State Building Code Commission reveals that 339 municipalities, 21.6 percent, in the state have building codes or regulations. Questionnaires indicated that 56 of 62 cities had codes or regulations, 63 of 950 towns (752 replied), and 220 of 555 villages (513 replied). The questionnaire was mailed to 1567 municipalities, replies received from 1327.

Construction outlook bright

Construction will likely continue at relatively high levels during defense mobilization said Thomas S. Holden, president of F. W. Dodge Corp. in a brochure, "Construction in a Defense Economy."

"The present prospect is that the American economy will within a reasonable period adjust itself to carrying as a normal overhead burden a military establishment of 3,500,000 armed men with proportionate facilities and equipment. The carrying of such a burden is likely to prove less difficult than many people have imagined it might be."

Pointing out that the country is in transition from a sub-normal national defense pattern to a postwar normal, Mr. Holden expressed the opinion that "any cutbacks in over-all construction volume that may take place during the current transition period are apt to be moderate, perhaps approximating the amount of cutbacks that might have come as a normal market reaction from the 1950 boom."

Defense production requirements will be largely met by increased production of basic materials, Mr. Holden said.

"Once the transition to normal postwar defense has been accomplished," he said, "regulations, limitation orders and controls can be greatly relaxed, if not eliminated. Expansion of the American economy will of necessity require construction in great volume and in wide variety of structural types."

In explaining a moderate view of how construction may prove to be curtailed by the defense program, Mr. Holden pointed out that "there has been a general tendency to expect a repetition of all the shortages and all the control patterns that were experienced in World War II. Many construction industry people have even expected a squeeze like the one that occurred in 1943 and 1944, following the 1942 peak of war construction. Fortunately, a more realistic view is beginning to prevail."

"At this moment it would seem sound to assume that the squeeze on construction activity this year would continue into 1952. If it is necessary to make any assumptions at this time concerning 1952 construction volume," he said, "it would be safer to assume for the year some decrease from 1951 volume, though such decrease is by no means a certainty."

But he pointed out that even such a decline from the all-time high figures of 1950 still represents large volume.

"Demand will continue strong," he said, "for military construction projects, defense plants, defense housing; also housing to meet needs of new families, for ordinary community facilities, and for commercial building projects deferred by reason of the M-4 order."

• • •

Rising costs make it a necessity . . . .

Effective the first of the JOURNAL Volume Year No. 23, September, 1951, the subscription price of the JOURNAL will be increased from $10 to $12 per year in the United States, U. S. possessions and North American countries; to $15 in all other countries.

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Continuous girder bridge features hinged piers

Hinged piers were incorporated in the continuous girder reinforced concrete bridge over the Zumbro River, Rochester, Minn. Combination structural steel and reinforced concrete columns for the piers made practical the use of a hinge at the bottom of each pier column. The columns were made as small as possible to provide maximum clear waterway. The bridge, crossing at an angle, is about 220 ft long with a 155 ft channel between walls. Center span girders are 82 ft; end span girders about 66 ft.

Honor Roll

February 1—March 30, 1951

Morgan jumps into lead spot

Thirteen of ACI's 5267 members made the Honor Roll this month for proposing 2 or more new members. Newlin D. Morgan, Sr., leads with 28 credits; Eddy Hernandez holds second place with 6½ and H. J. Gilkey has third place with 6 credits.

Newlin D. Morgan (Ill.)	28
Eddy Hernandez (La.)	6½
H. J. Gilkey (Ia.)	6
J. E. Jellick (Calif.)	5
Oscar A. Nunez G. (Venezuela)	3½
Frank Kerekes (Ia.)	2½
Fernan Rodriguez-Gil (Venezuela)	2½
Rafael Ruiz (Guatemala)	2½
Robert G. Deitrich (Md.)	2
L. G. Forrant (Fla.)	2
Samuel Hobbs (Calif.)	2
Howard Simpson (Mass.)	2
George D. Youngclaus (Calif.)	2

New Members

88 new members

ACI's membership total was 5267 on April 1 with 88 new member applicants received in March—37 Individual, 1 Corporation, 13 Junior and 37 Student. Illinois was top membership state this month with 25 new members. Received from outside the United States were 5 from Venezuela, 4 from Japan, 3 from Canada and 1 from Colombia.

Individual

Anderson, R. L., Bellaire, Texas
Armour, Cecil V., Los Angeles, Calif.
Byrd, James Cosby, Winnsboro, S. C.
Chester, W. F., Jr., Atlanta, Ga.
Cook, Rudyard M., Evanston, Ill.
Dise, John Robert, Arlington, Va.
Erb, Don, Long Beach, Calif.
Frewen, J. S., Walla Walla, Wash.
Harbeson, H. R., Nottawa, Mich.
Horvath, William B., New York, N. Y.
Johnson, Clair L., Pontiac, Mich.
Kokubu, Mastane, Tokyo, Japan
Lawrence, N. A., Edmonton, Alta., Canada
Mansur, Cline L., Tulsa, Okla.
McPhee, Angus N., Seattle, Wash.
Meredith, Diven, Thermal, Calif.
Miron, Adrien, Montreal, Que., Canada
Mizuno, Takaaki, Fukuoka, Japan
Moore, William J., Manchester, Conn.
Naruse, Katsutake, Tokyo, Japan
Olivares-Sosa, Caracas, Venezuela
Orr, Lesley K., Pickstown, S. D.
Porteus, J. H., Sewickley, Pa.
Ramirez, Angel Vicente, Yauco, Puerto Rico
Rosete-Gomez, Jesus, Caracas, Venezuela
Saffir, Herbert S., Miami, Fla.
Siebert, Emil, New York, N. Y.
Smithson, S. E., Milesburg, Pa.
Spilleès, Frank L., Denver, Colo.
Stevenson, Willard H., Ft. Wayne, Ind.
Strutz, Leonard J., Pickstown, S. D.
Tchernowitz, Jack K., New York, N. Y.
Tommerup, Carl C. H., Seattle, Wash.
Valonis, Edmund J., Chicago, Ill.
Van Erp, J. W. T., Cleveland, Ohio
Wicks, Charles H., Joliet, Ill.
Zeigler, M. G., Columbia, S. C.

Corporation

Pioneer Sand & Gravel Co., Seattle, Wash. (Claude L. Golding)

Junior

Biederman, Jack Carl, Calumet City, Ill.
Campilongo-T, Generoso, Caracas, Venezuela
Critton, Clarence M., Vallejo, Calif.
Davis, Fred, Napa, Calif.
Fabricius, Hans Eberhard, Coral Gables, Fla.
Ferrer-Fernandez, Diego, Miranda, Venezuela
Latenser, Nes, Omaha, Neb.
Morrison, Harold L., Edmonton, Alta., Canada
Mouradian, Archie, Longview, Texas
Oshima, Hiroshi, Yokohama, Japan
Pereira-Paiva, Orestes, Caracas, Venezuela
Richter, Robert W., Chicago, Ill.
Sandvig, Leo D., Pickstown, S. D.

Student

Bistransky, Joseph, Gary, Ind.
Bleck, Thomas F., Waukegan, Ill.
Bromley, Arthur L., Chicago, Ill.
Buck, William Adams, Decatur, Ill.
Cantor, Marvin J., Bronx, N. Y.
Ciha, Aloysius G., Champaign, Ill.
Correo, Luis Guillermo, Baton Rouge, La.
Costanza, Anthony J., Atlanta, Ga.
Cowling, Robert J., Champaign, Ill.
Dankert, Russell Arthur, Indianapolis, Ind.
Gaetani, Arthur L., Jr., Milton, Mass.
Gonzalez V., Jorge, Caracas, Venezuela
Guerra, Jose S., Barranquilla, Colombia
Hardtke, Charles F., Jr., Urbana, Ill.
Istrabadi, Rasoul M., Baton Rouge, La.
Jablonicky, Michael Peter, Milwaukee, Wis.
Jacobson, John A., Watertown, Mass.
Jaramillo, Peolbo, Baton Rouge, La.
King, John C., Danville, Ill.
King, Thomas T., E. St. Louis. Ill.
Linder, James A., Decatur, Ill.
McDonald, Charles James, Christopher, Ill.
More, Donald H., Urbana, Ill.
Mosher, Rollin V., Cuba, Ill.
Naughton, John Joseph, Chicago, Ill.
Nelson, David, New York, N. Y.
Pardo, Rafael, Baton Rouge, La.
Rabin, Zorack, Chicago, Ill.
Rothman, Marvin, Chicago, Ill.
Scaggs, John R., Collinsville, Ill.
Swank, E. Robert, Ft. Wayne, Ind.
Tanner, W. Harold, Lerna, Ill.
Tovar, Gonzalo, Baton Rouge, La.
Verkade, Andrew, Chicago, Ill.
Weeber, Hernando Q., Baton Rouge, La.
Williams, George J., Champaign, Ill.
Zerante, James J., Jr., Chicago Heights, Ill.

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Contributing Members give additional help to the Institute in bringing you the latest information on concrete. With the same member privileges enjoyed by Corporation and Individual Members, these 49 corporations who maintain 65 Contributing Memberships provide additional financial support for ACI work.

	Contributing Memberships
Allentown Portland Cement Co. (1936), Catasauqua, Pa.	1
Alpha Portland Cement Co. (1905), Easton, Pa.	1
Ash Grove Lime & Portland Cement Co. (1936), Chanute, Kan.	1
Bethlehem Steel Co. (1924), Bethlehem, Pa.	1
Calaveras Cement Co. (1941), San Francisco, Calif.	1
California Portland Cement Co. (1941), Colton, Calif.	1
Canada Cement Co., Ltd. (1916), Montreal, Que., Canada	1
Chain Belt Co. (1936), Milwaukee, Wis.	1
Consolidated Cement Corp. (1936), Jackson, Mich.	1
Dewey & Almy Chemical Co. (1944), Cambridge, Mass.	1
Dewey Portland Cement Co. (1936), Kansas City, Mo.	1
Dewey Portland Cement Co. (1936), Davenport, Ia.	1
Ford, Bacon & Davis, Inc. (1927), New York, N. Y.	1
General Portland Cement Co. (1936) (1944), Tampa, Fla.	2
Glens Falls Portland Cement Co. (1908), Glens Falls, N. Y.	1
Hercules Cement Corp. (1936), Philadelphia, Pa.	1
Huron Portland Cement Co. (1936), Detroit, Mich.	2
Jaeger Machine Co. (1937), Columbus, Ohio.	1
Keystone Portland Cement Co. (1941), Bath, Pa.	1
Koehring Co. (1944), Milwaukee, Wis.	1
Kosmos Portland Cement Co. (1916), Louisville, Ky.	1
Laclede Steel Co. (1921), St. Louis, Mo.	1
Lehigh Portland Cement Co. (1924), Allentown, Pa.	3
Lock Joint Pipe Co. (1925), Ampere, N. J.	1
Lone Star Cement Corp. (1938), New York, N. Y.	1
Louisville Cement Co. (1924), Speed, Ind.	1
Marquette Cement Mfg. Co. (1936), Chicago, Ill.	2
Medusa Portland Cement Co. (1916), Cleveland, Ohio	1
Missouri Portland Cement Co. (1936), St. Louis, Mo.	1
Monolith Portland Cement Co. (1941), Monolith, Calif.	2
National Ready Mixed Concrete Assn. (1937), Washington, D. C.	1
National Sand & Gravel Assn. (1937), Washington, D. C.	1
Nazareth Cement Co. (1916), Nazareth, Pa.	1
Northwestern States Portland Cement Co. (1936), Mason City, Ia.	1
Peerless Cement Co. (1936) (1937), Detroit, Mich.	2
Pennsylvania-Dixie Cement Corp. (1942), New York, N. Y.	1
Permanente Cement Co. (1941), Oakland, Calif.	3
Permanente Cement Co. (1941), Cupertino, Calif.	2
Raymond Concrete Pile Co. (1916), New York, N. Y.	1
San Antonio Portland Cement Co. (1926), San Antonio, Texas	1
Santa Cruz Portland Cement Co. (1941), Davenport, Calif.	1
Signal Mountain Portland Cement Div. (1936), Chattanooga, Tenn.	1
Southwestern Portland Cement Co. (1926), Los Angeles, Calif.	3
Stone & Webster Engr. Corp. (1920), Boston, Mass.	1
Superior Portland Cement, Inc. (1936), Seattle, Wash.	1
Trinity Portland Cement Div. (1936), Dallas, Texas	1
Turner Construction Co. (1919) (1920), New York, N. Y.	2
Universal Atlas Cement Co. (1914) (1937), New York, N. Y.	2
Waylite Co. (1942), Chicago, Ill.	1
Whitehall Cement Mfg. Co. (1908), Cementon, Pa.	1
Worthington Pump & Machinery Corp. (1937), Dunellen, N. J.	1

NPA construction controls outlined

"Basically, the question in construction today is structural steel," said Manly Fleischmann, administrator, National Production Authority, in speaking April 11 about material shortages for construction and the part the Controlled Materials Plan will play in defense mobilization.

To provide materials for essential construction, such as highways, schools, hospitals and plant expansions, he said that a plan similar to that used in World War II probably will be followed.

Large industrial projects will be reviewed and an allotment made to those needed in the defense program. Allotments will have to be provided for essential school buildings, or they will be in a free area. Roads have a similar problem. He said that new construction will be curtailed but maintenance is a necessity. Such work may operate in the free area or some steel may be set aside. NPA does not plan to program residential construction now.

"Decisions, to some extent, depend on information from areas we know we have to program," said Mr. Fleischmann. "If there is more steel available than presently seems likely, perhaps we can leave this open end a little longer than otherwise."

The Controlled Materials Plan will begin July 1 and is based on the World War II plan. It will differ in one respect in that it will be an open-ended plan. The exact extent of that open area has not been finally determined, Mr. Fleischmann said.

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Big scoop

A 5-cu yd materials handling scoop developed by Mixermobile Manufacturers is claimed to be the largest of its kind in the construction industry. It can be used for elevating and placing concrete and loading and shoveling dirt and bulk materials. The tires are as tall as an average man and the machine weighs more than 18 tons. It is powered by a 250 hp diesel engine. The track tilt has hydraulic control and the carriage can be operated with the track in any position.—Mixermobile Manufacturers, 8027 N.E. Killingsworth, Portland 20, Ore.

Tapping prestressed pipe

Methods used in tapping prestressed concrete steel cylinder pipe are illustrated in a 4-page leaflet released by Price Brothers Co. It illustrates step-by-step procedure.—Price Brothers Co., 1932 East Monument Ave., Dayton, Ohio.

Rack simplifies block handling

Handling of concrete blocks can be simplified with a new type rack states Equipment Manufacturing, Inc. The rack eliminates individual block handling and makes multiple handling practical in drying and curing, storing and shipping. Of welded structural steel construction, the rack holds 12 pallets of the size used for casting four blocks. The bottom level is raised from the floor to permit lifting and transporting by fork or platform truck from casting to storage.—Equipment Manufacturing, Inc., 21550 Hoover Road, Detroit 5, Mich.

Shovel for tunnel operations

Especially designed for tunnel construction, the new Underground Lodover manufactured for International Harvester tractors by Service Supply Corp. is an adaptation of the standard Lodover. The manufacturer states that the combination overhead and front-end tractor loader loads overhead in a 14-ft shaft. It is claimed that elimination of turns in loading often steps up production as much as 50 percent. Equipped with a 1¼-yd heavy duty rock bucket, the unit can dig its own sloping ramp in and out of the tunnel or shaft.—Manufacturing Division, Service Supply Corp., Philadelphia 32, Pa.

Waterproof coating

A new waterproof cement coating has been put on the market by Durepel Corp. The manufacturer states that the coating contains no organic material but is a combination of mineral and metallic components. Preparation of surfaces and Durepel's use is described in a recent leaflet.—Durepel Corp., 10 East 43rd Street, New York 17, N. Y.

Membership certificates

Repeated but infrequent requests for membership certificates indicate that enough members may want them to warrant having them made. At its fall meeting, the Board of Direction authorized the Secretary to determine through this notice whether or not the Institute would be justified in going to the expense of having the necessary work done.

It is estimated that certificates suitable for framing can be furnished ACI Members at a cost of not over $3.50. Those interested are requested to clip the attached coupon and mail it to Institute headquarters. *Please do not send money until billed.* If there are not enough interested, the Institute would not be justified in supplying certificates to a few at a loss.

American Concrete Institute
18263 W. McNichols Road
Detroit 19, Mich.

Gentlemen: Please place an order for one membership certificate to be furnished provided enough members buy them to make it worthwhile at a cost not to exceed $3.50.

Signed.....

Pipeline concrete

A revised bulletin, No. 51-31, on Rex Pumpcrete has been published by Chain Belt Co. Its 24 pages describe placing concrete by pump and pipeline. It indicates the type of jobs on which pumping concrete is advantageous, what type of mixes pumps will handle, and the height and distance requirements of a placing job.—Chain Belt Co., 1600 W. Bruce Street, Milwaukee 4, Wis.

Form construction

Construction data for form systems is presented in recent bulletins released by Symons Clamp and Mfg. Co. The leaflets illustrate form hardware, standard panel forms and assembly procedure. Directions are given and illustrated for form construction at corners and pilasters, use of walers, heavy pressure forms, bracing and runway scaffolds.—Symons Clamp and Mfg. Co., 4249 Diversey Ave., Chicago 39, Ill.

On-the-spot testing

To test concrete structures in the field, Ernst Schmidt, a Swiss engineer, has developed a novel instrument for obtaining a measure of the compressive strength by observing the rebound of a spring-propelled hammer off the concrete surface.

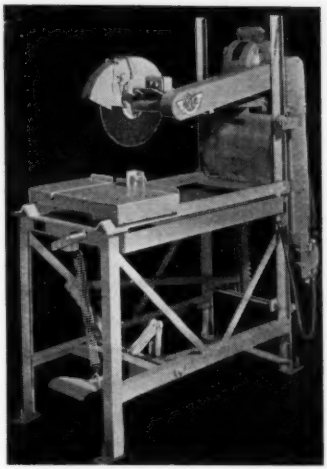
A lightweight steel hammer, free to travel in a tubular frame, is drawn against spring tension by a trigger. The base of the frame is firmly pressed against the concrete and when the hammer is fully drawn, the trigger is released, allowing the spring to drive the hammer against the concrete. A small sliding pointer indicates the rebound on a scale. The amount of rebound, *R*, on the scale is a measure of the compressive strength of the concrete as determined by the hardness at the point tested, says the inventor.

The compressive strength can be estimated to within 15 to 20 percent by 6 to 10 rebound readings, taking about one minute.

The relationship between the rebound number and compressive strength was investigated by the Swiss Federal Testing Laboratory. About 550 specimens from various types of jobs were subjected to ten impacts each with the test hammer and the mean value compared with compressive strength as obtained in a testing machine.

Before testing different types of concrete, laboratory correlation between compressive strength and rebound number would be necessary.

The inventor says that the test hammer is not a substitute for control test cylinders or core tests, but a supplemental tool. It can be used to check the uniformity of concrete quality in the actual structure. It is also useful in determining the rate at which concrete reaches a required strength for determining the safe time for removal of formwork and shoring.—Arthur R. Anderson, Consulting Engineer, 110 Woodbury Ave., Springdale, Conn.



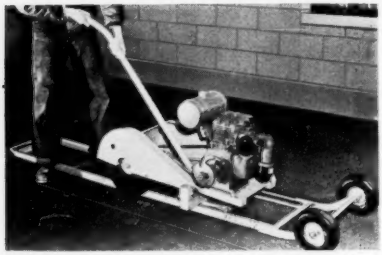
Masonry saw

The Supreme Equipment Corp. has developed a new masonry saw, the "Supreme Speed Kut Masonry Saw." A waist-high handerank controls and fixes the positive self-locking wheel-positioning mechanism. The manufacturer says that by turning the crank the machine may be adjusted for cutting any commercial size block, flat or endwise. The machine is powered by an enclosed, heavy duty, 1½ hp motor.—Supreme Equipment Corp., 12415 Euclid Ave., Cleveland 6, Ohio

Pavement cutter

A pavement cutter developed by Joy Manufacturing Co. has two large blades, carrying Sulmet tungsten carbide tipped bits, that saw parallel 2-in. slots in pavement, 18 to 54 in. apart. The strip of paving between slots can then be removed in chunks with a backhoe or similar equipment. Preliminary testing is said to have indicated that the cutter will triple the speed and halve the cost of pavement removal.

The machine is said to be capable of slotting to depths of 15 in. in almost any kind of paving material. It can cut at speeds up to 4 ft per min., depending on thickness and hardness of material. The manufacturer says the rate of advance through 8-in. concrete with mesh reinforcement is 3 to 4 ft per min.—Joy Manufacturing Co., Henry W. Oliver Building, Pittsburgh 22, Pa.



Concrete saw

A gasoline powered concrete cutting saw has been developed by Martin Fireproofing Corp. It is similar to their electric powered saw. The manufacturer says that guide rails on the saw give a smooth edged, straight cut and eliminate blade breakage caused by wobble or application of off-center pressure.—Martin Fireproofing Corp., P. O. Box 27, Kenmore Station, Buffalo 17, N.Y.

Prestressed concrete steel-cylinder pipe

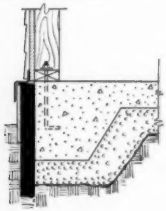
A well-illustrated booklet published by Price Brothers Co. tells how prestressed concrete steel-cylinder pipe is made. Of particular interest is the explanation of how the steel cylinder and concrete core are prestressed to form an elastic structure and how the pipe handles high pressures with a minimum of stress.—Price Brothers Co., 1932 E. Monument Ave., P. O. Box 825, Dayton 1, Ohio.

Material handling

Kalamazoo Mfg. Co. has announced that the "Kal-Truk" for handling various types of construction materials can now be equipped with a dump control on the front end gate. It is claimed that the control door allows easier placing of concrete, or other material, into small or narrow openings. The four-wheel truck has a load capacity of ⅜ cu yd or 3000 lb and is powered by a 13 hp engine.—Kalamazoo Mfg. Co., Kalamazoo 24F, Mich.

How to cure a cold floor

Cold uninsulated concrete slab floors in basementless homes can be corrected even if the house is already built, says Celotex Corp. Cold floors can be made warmer by placing two layers of 1-in. Flexcell board around the slab.



To provide space for application of insulation board, it is necessary to trench around exterior 12 in. or more below grade. The board is adhered to the face of asphalt primed concrete with cold asphalt mastic or hot asphalt. Metal flashing is recommended over the top. For protection above grade and neat appearance, asbestos cement board is cemented against the insulation board.

Flexcell, a cane fiber board saturated with an asphaltic compound, is used also as expansion joint filler in concrete pavement.—Celotex Corp., 120 South LaSalle Street, Chicago 3, Ill.

Tunnel mixers

Bulletin R-1700-B5 from Worthington Pump and Machinery Corp. describes dual drum tunnel mixers. Specifications are listed and features and attachments illustrated.—Worthington Pump and Machinery Corp., Harrison, N. J.

Foundation structures

Drilled and poured foundation structures are described in a well-illustrated brochure published by Casey and Case Foundation Co.

Types and applications of foundation structures, pressure grouting, drainage, de-watering, shoring and subsurface soil exploration are illustrated in case history fashion. Of particular interest are the tables on volume of concrete required for caisson bells of different diameters in relation to various size shafts.—Casey and Case Foundation Co., P. O. Box 151, Maywood, Calif.

Impact breaker

A heavy duty impact breaker, the PMCA Impact Master, has been developed by Pettibone Mulliken Corp. The manufacturer states the breaker controls the breaking operation and directs the flow of material through the machine to produce a uniformly graded cubical aggregate.

Two rotor hammers rotate in the same direction at 550-1000 rpm. Size of finished material is governed by the speed of the rotors and adjustments of a stripper bar and a lower screen gate. The machine has a reduction ratio between 32:1 and 40:1, states the manufacturer. It will accept quarry run rock up to 50 in. long. The machine has a capacity of 250 tons per hour of −2½ in. average limestone.—Pettibone Mulliken Corp., 4700 West Division Street, Chicago 51, Ill.

Combats lost circulation in oil drilling

Strata-Seal, a lightweight additive to combat and prevent lost circulation of drilling mud, has been developed by Great Lakes Carbon Corp. for use in the oil fields. A recent leaflet describes the material and its applications.—Strata-Crete Sales, Great Lakes Carbon Corp., 5845 Atlantic Ave., Long Beach 5, Calif.

Water meter

Auto-Stop water meters, which automatically control the amount of water delivered to ready-mixed concrete batches, are described in the new Bulletin 562 released by Neptune Meter Co. The meters are operated by pushing buttons to set the number of gallons desired, turning a reset handle and flipping the valve handle. The tripping device will automatically cut off the flow when the exact quantity is delivered to the mix.—Neptune Meter Co., 50 West 50th Street, New York 20, N. Y.

Placing concrete with vibrators

Mall Tool Co. has published a booklet "Placing Concrete with Mall Concrete Vibrators." The well illustrated booklet cites the ACI Manual of Concrete Inspection as a reference. Benefits of vibration are listed, as well as suggested trial mixtures, capacities of vibrators for various classes of construction, placing instructions and recommended practices in the use of vibrators.—Mall Tool Co., 7740 South Chicago Ave., Chicago 19, Ill.

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 22 are currently available at prices indicated. Please order by title and title number.

SOME OBSERVATIONS ON THE USE OF REINFORCING STEEL IN CONCRETE PAVEMENTS.....47-1

Price 35 cents.

BENGT F. FRIBERG—Sept. 1950, pp. 1-16 (V. 47)

The paper contains a few accumulated observations from a review of literature on the evolution of reinforced concrete pavement designs, with reference to various systems of reinforcement which have seen extended use. Typical structural concrete pavement failures are described. Trends in design and reinforcement practice are shown, with special reference to recent developments. Pertinent findings of a few published pavement surveys are cited, with special attention to the 900-mile Louisiana pavement survey made in 1945. The need for performance information and additional research is stressed.

TESTS OF PAPER MOLDS FOR CONCRETE CYLINDERS.....47-2

Price 35 cents.

ROBERT A. BURMEISTER—Sept. 1950, pp. 17-24 (V. 47)

Concrete test cylinders cast in a new type paper mold had a compressive strength lower than that predicted for the concrete mix used. Investigation showed that cracks and mechanical injuries to the outer shell of the concrete cylinder caused by movement of the paper stock during the first 24 hours of curing reduced the strength of the cylinder. To a lesser degree this was true also of the paraffined paper molds in common use for casting test cylinders, specimens cast in both types of paper molds showing lower strengths than test cylinders from the same mix cast in steel molds.

ADMIXTURES IN CONCRETE.....47-3

Price 60 cents.

W. T. MORAN, F. H. JACKSON, BRUCE E. FOSTER and T. C. POWERS—Sept. 1950, pp. 25-52 (V. 47)

Five papers by members of ACI Committee 212, Admixtures, are, because of their common general subject, presented together.

Various admixtures are discussed briefly as an introduction to more detailed treatment of air-entraining materials. The relative merits of admixtures and interground agents are considered. Optimum ranges of air content for different structural uses are given with particular reference to pavements.

The advantages and disadvantages of several types of admixtures used in the fabrication of various concrete products, such as building block, cast stone, pipe, cribbing and curbing, are discussed. The admixtures considered are classified into the following groups: accelerators, air-entraining agents, gas-forming agents, water repellent agents, and workability agents.

The factors affecting bleeding characteristics and workability of fresh concrete are reviewed and the effect of admixtures on these properties is assessed.

Present knowledge of admixtures in counteracting alkali-aggregate reactions is reviewed. It is emphasized that further studies may revise thinking in this field.

The effectiveness of various concrete admixtures in inhibiting the capillary flow of water and the flow of water under pressure is considered. The types of admixtures included in the discussion are accelerators, soaps, butyl stearate, finely subdivided dry materials, mineral oil, workability agents, and a miscellaneous group of proprietary compounds.

IMPROVED SONIC APPARATUS FOR DETERMINING THE DYNAMIC MODULUS OF CONCRETE SPECIMENS.....47-4

Price 35 cents.

C. E. GOODELL—Sept. 1950, pp. 53-60 (V. 47)

After a brief introduction to sonic testing and a description of commercial equipment, the apparatus built for the Michigan State Highway Department is discussed. Reliable results can be obtained by an unskilled worker with this compact equipment which has twice the frequency band spread of the usual oscillator. A wiring diagram of the sonic apparatus is included.

ANALYSIS OF THREE-DIMENSIONAL BEAM-AND-GIRDER FRAMING.47-5

Price 35 cents.

PHIL M. FERGUSON—Sept. 1950, pp. 61-72 (V. 47)

The beam-and-girder floor with some beams carried directly by columns and others supported on girders is cited as a practical problem in frame analysis that must include the torsional stiffness of the girder. Curves showing how moment coefficients vary with this torsional stiffness are developed for a few simple cases in interior panels. These show the weakness of rule-of-thumb methods.

A practical calculation form is set up for use with the moment distribution method in solving three-dimensional problems of this type.

Proposed Revision of SPECIFICATIONS FOR CONCRETE PAVEMENTS AND BASES (617-44).....47-6

Price 35 cents.

REPORT OF COMMITTEE 617—Oct. 1950, pp. 93-116 (V. 47)

New specifications incorporated include those covering air entrainment, removal of forms, premolded joint fillers and joint filling materials and method of placing reinforcement. Definitions have been added under soil foundation preparation and other parts of this section have been revised.

LINEAR TRAVERSE TECHNIQUE FOR MEASUREMENT OF AIR IN HARDENED CONCRETE.....47-7

Price 35 cents.

L. S. BROWN and C. U. PIERSON—Oct. 1950, pp. 117-124 (V. 47)

The method described for the determination of air in hardened concrete permits the examination of 6 x 8-in. and 6 x 10-in. random plane face-ground hardened concrete specimens which more truly represent the aggregate and air voids in the actual concrete than smaller specimens. The construction and use of the instruments are discussed and results of tests are given. Because of the time and equipment necessary to measure air content by means of the integrator, it is not adaptable to field use. However, as a laboratory tool it provides a means for quick and accurate determination of total air.

INFLUENCE OF THE QUALITY OF MORTAR AND CONCRETE UPON CORROSION OF REINFORCEMENT.47-8

Price 35 cents.

RACHEL FRIEDLAND—Oct. 1950, pp. 125-140 (V. 47)

In tests to determine the influence of the quality of mortar and concrete upon corrosion of reinforcement the variables studied were cement content, water-cement ratio, consistency, grading and depth of cover. The specimens, stored in moist air or exposed to weather, were tested up to the age of 2 years.

The results indicate that consistency has a pronounced effect upon the protective value of mortar and concrete, and that there appears to exist an "optimum consistency" at which the quantity of rust is practically unaffected by time. It was also found that the usual cement contents in reinforced concrete have only a limited effect upon corrosion. It is concluded that water-cement ratio does not in itself control the rate of corrosion of reinforcement.

PROPER SAND GRADING IMPROVES MASS CONCRETE.....47-9

Price 35 cents.

THOMAS B. KENNEDY—Oct. 1950, pp. 141-152 (V. 47)

Two series of concrete mixtures were designed using 6-in. traprock coarse aggregate and eight separate gradings of natural sand ranging in fineness modulus from 3.60 to 1.35. Tests were made of the plastic concrete, and specimens were cast for tests of compressive strength, resistance to freezing and thawing, and drying shrinkage. Both series of concrete mixtures had a cement content of 2.5 bags per cu yd; one had a normal air content—4½ ± 1½ percent in the portion of the mixture passing the 1½-in. sieve—and the other had a high air content—10 ± 2 percent in the portion of the mixture passing the 1½-in. sieve. Tests indicate that good durability in freezing and thawing can be obtained within the normal air content range with fineness modulus between 2.50 and 2.90. With increased air content, however, the fineness modulus range can be increased to extend from 1.58 to 3.24. Compressive strength was generally affected adversely by increased air content, but not to a serious degree. Drying shrinkage was less with normal air content mixes than with high air content mixtures. It was least when a fineness modulus of 2.52 was used, little difference being apparent between the high and normal air content mixes with this fineness modulus. The air-entraining admixture requirement increased greatly as the fineness modulus of the sand decreased. The water ratio also tended to increase with decreasing fineness modulus.

WATER-SOLUBILITY OF ALKALIES IN PORTLAND CEMENT.....47-10

Price 35 cents.

J. L. GILLILAND and T. R. BARTLEY—Oct. 1950, pp. 153-160 (V. 47)

In an effort to show correlation of soluble alkalies with alkali-aggregate reaction, the authors hydrated a number of cements for periods up to 90 days and analyzed water extracts of the ground hydrated cement. However, the correlation with expansions of mortar bars prepared with reactive aggregate was not improved by considering water-soluble alkalies rather than total alkalies.

The rate at which the alkalies become water-soluble in hydrating cement indicates that the alkali-bearing phases in cement hydrate quite readily.

NEW PRESTRESSING METHOD UTILIZES VACUUM PROCESS.....47-11

Price 35 cents.

K. P. BILLNER—Oct. 1950, pp. 161-176 (V. 47)

The method outlined here was developed to simplify prestressing of concrete to make it generally adaptable to American ways of construction. It eliminates costly anchorages; uses large diameter wires (⅜-in. diameter now available on the market), instead of the customary ⅛-in. diameter wire, thus greatly reducing the number of wires required; prestresses all the wires in the building element simultaneously and simplifies forming. A simplified method of design calculations for prestressed concrete and the result of tests of a beam so designed are included.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF MORTAR BY PNEUMATIC PRESSURE 47-12

Price 35 cents.

REPORT OF COMMITTEE 805—Nov. 1950, pp. 185-196 (V. 47)

This proposed ACI Standard presents briefly the advantages and disadvantages of pneumatically-placed mortar and establishes recommended practices for placing and mixing shotcrete, qualifications and duties of workmen, preparation of surface before shotcreting, reinforcing, sequence of application, and other items involved in good shotcreting.

DETERMINING OPTIMUM CROSS SECTIONS FOR PRESTRESSED CONCRETE GIRDERS.....47-13

Price 35 cents.

FRED J. UZIEL—Nov. 1950, pp. 197-212 (V. 47)

General solutions are presented for selecting economically optimum cross sections for prestressed concrete flexural members. Such a solution for simply-supported girders of rectangular cross section, for any span and load, assuming the wires prestressed after setting of the concrete and full dead weight to act during the prestressing operation, leads to a design procedure which, in the case considered, is extremely simple and rapid to apply. It also permits comparisons and studies related to the critical cross section to be made in a more general and conclusive manner. For cross sections other than rectangular, the number of variables making the solution more indeterminate in nature, a similar direct solution is not obtainable. However, a rapid way of obtaining the minimum areas of concrete and steel required is suggested.

STEAM CURING PROTECTS WINTER CONCRETING.....47-14

Price 35 cents.

C. O. CRANE—Nov. 1950, pp. 213-216 (V. 47)

The use of live steam for protecting newly placed concrete from freezing weather and for providing initial curing has resulted in excellent concrete in the Enders Dam spillway at no greater cost than less desirable dry heating methods. This brief paper describes in detail the methods used.

ANALYSIS OF CONTINUOUS CIRCULAR CURVED BEAMS.....47-15

Price 35 cents.

BECLA VELUTINI—Nov. 1950, pp. 217-228 (V. 47)

Continuous circular curved beams can be analyzed easily by the moment distribution method if both bending and torsional end couples are considered. Formulas and tables are presented for circular curved beams of constant cross section that give the relations between the end moments and end torques and the corresponding rotations of the end sections. A proposed method of procedure is illustrated in which the bending end couples are kept separate from the torsional couples. The mathematical operations are not difficult as the convergence is rapid, but attention must be paid to the sign convention which must be definite and easy to apply.

SHEAR RESISTANCE OF TILE-CONCRETE FLOOR JOISTS.....47-16

Price 35 cents.

J. NEILS THOMPSON and PHIL M. FERGUSON—Nov. 1950, pp. 229-236 (V. 47)

Tests on certain types of tile-concrete joists indicate that the tile webs are more effective in resisting diagonal tension than is indicated by the current ACI Building Code specification. Stagger of tile joints appears to be unnecessary, since they do not seem to be planes of weakness insofar as diagonal tension is concerned. The tile reduces the deflection of the joist.

SOLUTION OF DIFFICULT STRUCTURAL PROBLEMS BY FINITE DIFFERENCES.....47-17

Price 35 cents.

ALFRED PARME—Nov. 1950, pp. 237-256 (V. 47)

Finite differences can be applied to the solution of those structural problems in which the physical relationships are expressed as a differential equation. Essentially, the technique employed consists of replacing the derivatives of the differential equation by its central difference equivalent. The problem is thus reduced to the simple task of solving a system of simultaneous linear algebraic equations. The numerical computation involved in the procedure is considerably reduced by two devices. First, the number of equations necessary to attain sufficient accuracy is reduced by an evaluation of the error introduced in substituting central differences for derivatives. Secondly, the solution of simultaneous equations is speeded by a systematic rapid tabulation of easily determined values. The procedure is applied to the design of a sheet pile wall, elliptical dome and skewed bridge to illustrate the scope and simplicity of the method.

Proposed Revision of BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-47).....47-18

Price 35 cents.

REPORT OF COMMITTEE 318—Dec. 1950, pp. 269-276 (V. 47)

Proposed changes decrease the allowable bond stress in plain bars (including the old types of deformed bars) and increase the allowable bond stresses for the new types of bars over those previously allowed for the old types. Top bars, those having more than 12 in. of concrete under them, are assigned lower bond stresses than bars in other positions. All plain bars must be hooked, which corresponds to special anchorage under the old provisions. The new bars develop sufficient anchorage by bond alone to correspond to special achorage with the old type bars. Consequently, all bars under the new provisions correspond to those with special anchorage under the old provisions.

CURING CONCRETE PAVEMENTS WITH MEMBRANES.....47-19

Price 35 cents.

C. C. RHODES—Dec. 1950, pp. 277-296 (V. 47)

To provide data to assess the advisability of continuing membrane curing of concrete pavements as an alternate method, laboratory and field tests were made to compare the effect of storage conditions on the physical properties of concrete, warping, temperature control, and strength and abrasion resistance of concrete cured with membranes and with wet burlap. A survey of pavements cured with clear membranes in spring and summer showed that cracking, when it occurred at all, was found predominantly in pavements laid in the morning hours. Comparing white-pigmented membranes with the usual wet-curing conditions in the field it was found to be efficient, practicable and about half as expensiv s wet curing under the same conditions.

BLADE CHANGES IMPROVE TILTING MIXER.....47-20

Price 35 cents.

GLENWAY MAXON—Dec. 1950, pp. 297-300 (V. 47)

Recent experiments, as well as earlier studies, on changing the blading of tilting concrete mixers so as to improve the quality of the mixed concrete are described. The evolution of the blade shapes and the effect of these changes on the path of the materials through the mixer are illustrated.

PROTOTYPE PRESTRESSED BEAM JUSTIFIES WALNUT LANE BRIDGE DESIGN.....47-21

Price 35 cents.

GUSTAVE MAGNEL—Dec. 1950, pp. 301-316 (V. 47)

The tests made on a prestressed concrete beam of 154 ft 8 in. span, identical to the beams of the main span of the Walnut Lane Bridge in Philadelphia, more than justified the adoption of prestressed concrete for the bridge. The test methods and results are described. The test beam exhibited a safety factor against cracking of about 2, which would be far lower for a reinforced concrete beam. The factor of safety against complete failure was about the same as for reinforced concrete while the deflection was less. It is possible to use prestressed concrete for beams where structural steel can not be used due to excessive deflection. The tests proved that the Walnut Lane Bridge will have an exceptional degree of safety with less weight and greater durability than would be possible with reinforced concrete, as well as being considerably cheaper than the conventional solution.

FINISHING AND CURING: A KEY TO DURABLE CONCRETE SURFACES.....47-22

Price 35 cents.

MYRON A. SWAYZE—Dec. 1950, pp. 317-332 (V. 47)

After a comparison of past and present pavement curing and finishing techniques the significance of timing and character of finishing and the timing and mode of curing are discussed. Laboratory tests are cited to show the effect of time of finishing and curing on surface durability to freezing and thawing. It is recommended that all concrete exposed to frost contain entrained air, have a low water-cement ratio and be thoroughly compacted after placing. A finishing and curing procedure is suggested which is adapted to the ambient conditions and to the hydration needs of the cement.

ECONOMY THROUGH BETTER CONTROL OF REINFORCING STEEL.....47-23

Price 35 cents.

F. TESSITOR and P. ROSEWARNE—Dec. 1950, pp. 333-340 (V. 47)

Difficulties and troubles encountered through the use of reinforcement steels not in strict accordance with present-day specifications are discussed. Data are presented to show the possibility of alleviating the situation for engineers, producers and contractors. Simplification of materials requirements, liberalization of code requirements to permit hot bending of bars and permanent identification of grade of steel would, in large measure, permit designers to apply reinforcement to structures in a more effective manner.

Proposed Revision of MANUAL OF STANDARD PRACTICE FOR DETAILING REINFORCED CONCRETE STRUCTURES (ACI 315-48).....47-24

Price 35 cents.

REPORT OF COMMITTEE 315—Jan. 1951, pp. 349-352 (V. 47)

Changes are proposed in bar designations to conform to the numbered designation of the U. S. Department of Commerce and in all drawings to agree with new bond values and anchorage details for new-style deformed bars. Editorial changes in the text of the Standard are outlined.

COARSE-GROUND CEMENT MAKES MORE DURABLE CONCRETE.....47-25

Price 35 cents.

HAROLD W. BREWER and RICHARD W. BURROWS—Jan. 1951, pp. 353-360 (V. 47)

A test procedure for mortar ring specimens is described. Rings containing coarse-ground cement shrunk less and showed greater resistance to freezing and thawing and to outdoor exposure than those containing fine-ground cement. These laboratory tests indicate that coarse-ground cement produces more durable concrete han fine-ground cement.

SIMPLE EQUIPMENT ECONOMICALLY EXPLORES PRESTRESSING..47-26

Price 35 cents.

MARVIN L. MASS and JACK R. JANNEY—Jan. 1951, pp. 361-364 (V. 47)

A tubular grip utilizing type metal to hold the prestressing wires is described. Other equipment includes a framework against which the prestressing force is applied and adjustable formwork for experimental beams. The equipment is so simple that it can be easily and inexpensively built to permit study of prestressed reinforced concrete in the small laboratory.

LABORATORY TESTS OF SPACED AND TIED REINFORCING BARS....47-27

Price 35 cents.

WILLIAM T. WALKER—Jan. 1951, pp. 365-372 (V. 47)

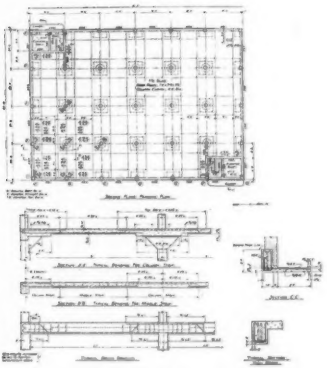
Tests of beam and pull-out specimens containing spaced and tied reinforcing arrangements indicated that little or no advantage would be obtained by spacing deformed

Manual of Standard Practice for Detailing Reinforced Concrete Structures

(ACI Standard 315-48)

for the

Designer
Detailer and
Student



$3.00 (to ACI Members—$1.75)

The **Manual** presents accepted methods and standards for preparing drawings for the fabrication and placing of reinforcing steel. The use of these methods and standards results in better reinforced concrete construction and also simplifies and reduces the amount of work ordinarily required to prepare such drawings.

The fundamental purpose of all "engineering drawings" is to enable the builder to transform the designer's calculations and design into the structure he has conceived. These drawings give the information necessary for building forms and placing concrete and for preparing the "placing drawings" (sometimes called "detail drawings") which must contain the information necessary for fabricating the reinforcement and placing it in the forms.

The **Manual** illustrates typical engineering and placing drawings prepared according to the standards. The designs are not intended as standard designs, although in general they meet the requirements of the American Concrete Institute's "Building Code Requirements for Reinforced Concrete" (ACI Code). They are intended to show **standard detailing methods.**

American Concrete Institute **18263 W. McNichols Rd.** **Detroit 19, Mich.**

The Detailing Manual includes—

Drawing standards—Standards and comments on size of sheet, layout, scales and symbols

Marks—Discussion of systems of marking members and reinforcement.

Schedules—Discussion of types of schedules and recommendations for their use.

Engineering drawings—Information required on drawings and best methods of showing it.

Placing drawings—Best methods of showing necessary information.

Fabricating detail standards—Standards for bending, hooks, stirrups, typical bar bends, slant lengths, beam widths, spirals, ties, column verticals, splices and dowels.

Notes to designers and detailers—Attention is called to points which frequently are overlooked or which cause trouble. Conformance with the suggestions given eliminate most difficulties of interpretation, fabrication and construction.

Shop procedure—Brief description of steps in the shop.

Warehouse stock—Notes material usually carried in stock.

Tolerances—Standard tolerances in dimensions of fabricated reinforcement.

Extras—Basis for extra charges for material and fabrication.

Welded wire fabric—Data on fabrication, size limitations, detailing and common styles.

Accessories—Standard accessories and their use.

Typical drawings—21 typical engineering and placing drawings are shown for various structures to illustrate the use of the standards and methods advocated in the manual. A short discussion of the important points accompanies each drawing.

Typical drawings

The engineering drawings used in the manual were selected from plans of actual structures but have been modified in some details to conform more closely with the ACI Code and to illustrate recommended methods of presenting the information. Shop prepared placing drawings accompany each engineering drawing.

- Concrete joist floor
- Flat slab floor
- Beam and girder floor
- Two-way slab and beam floor
- Foundations
- Columns
- Architectural concrete building
- Wall details
- Stair details
- Circular tank
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reinforcing bars at splices. Pull-out tests showed that for deformed bars, placed vertically, in which interlocking of lugs could take place there was a slight increase in strength due to tying the bars. Beam tests showed no significant difference between spacing and tying.

Twelve beams containing three different types of deformed bars either spaced or tied and extending the entire length of the specimens were tested. The sixty vertical pull-outs tested also contained three different types of deformed bars. Half of these were made with the bars spaced and half with the bars tied together. The reinforcing arrangement brought only one bar out of the specimens for pulling; the other was allowed to bear on the base plate.

Spacing of the bars in both beams and pull-outs conformed to the joint code minimum spacing requirement of 2½ diameters center to center with a minimum clear distance of 1½ times the maximum size of coarse aggregate.

FINISHING AIR-ENTRAINING CONCRETE PAVEMENTS.....47-28

Price 35 cents.

CHARLES W. ALLEN—Jan. 1951, pp. 373-376 (V. 47)

Based on a survey of state highway department practices, common difficulties in finishing air-entraining concrete pavements are enumerated. Practices in adjusting transverse finishing machines are given and revised finishing practices are discussed. Delayed finishing, as specified in several states, is considered in connection with the riding qualities of the finished surfaces. On the basis of experience in Ohio it is concluded that smooth pavements can be built of air-entraining concrete without delayed finishing.

EFFECTS OF LATERAL LOADS ON ARCHES.....47-29

Price 35 cents.

JAMES P. MICHALOS—Jan. 1951, pp. 377-388 (V. 47)

The effects of lateral loads on arches and the possible magnitude of these effects are considered. An influence table and influence lines are presented for moments and shears in unbraced parabolic arch ribs of constant cross section. These values are for several ratios of rise to span and several ratios of bending to torsional stiffness. The effect of haunching is studied and its possible importance is assessed. Procedures are presented for drawing approximate and exact curves of moments for unbraced arch ribs and for arch ribs braced with struts normal to the ribs. Numerical studies are included.

GETTING MORE FOR OUR CONCRETE DOLLAR.....47-30

Price 35 cents.

I. E. MORRIS—Jan. 1951, pp. 389-396 (V. 47)

A concrete slab with alternating horizontal elements connected by sloping elements is described and its design is worked out. The system is intended primarily for roofs where an exposed ceiling is desired for economy. It may be used to span considerable distances, either as a simple span or as a series of continuous spans. The members are designed to use both concrete and steel to maximum advantage.

FACTORS INFLUENCING CONCRETE STRENGTH.....47-31

Price 35 cents.

WALTER H. PRICE—Feb. 1951, pp. 417-432 (V. 47)

The effect of mix proportions, type and brand of cement, availability of moisture for curing, accelerators and curing temperatures on the rate and potential strength development of concrete are discussed. The influence of rate and frequency of load applications, dimensions of test specimens and lateral restraint on the indicated strength are also discussed, and information is furnished on the variations in strength which might be expected on a typical job. Compressive, tensile, flexural, bond and shearing strengths are compared, and the strengths of control cylinders are compared with the strengths of cores drilled from structures at later ages. Information is also furnished on strength loss from freezing and thawing and alkali-aggregate expansion.

SONISCOPE TESTS CONCRETE STRUCTURES.....47-32

Price 35 cents.

E. A. WHITEHURST—Feb. 1951, pp. 433-444 (V. 47)

The Soniscope, an instrument which measures group velocities through as much as 50 ft of concrete, was used for the field testing of 13 bridges, one navigation lock, 14 dams and five highway pavements in 12 states. Repeated tests permitted study of changes in the condition of the concrete and the development of group velocities indicating the condition of the structure. The value of the results increases with knowledge of the materials, mix design, method of placement and other characteristics of the structure being tested.

EFFECTIVE SEALING OF CONCRETE PAVEMENT JOINTS.....47-33

Price 35 cents.

H. F. CLEMMER—Feb. 1951, pp. 445-448 (V. 47)

The development of cork, metal and rubber joint sealers is traced and work with thermoplastic and cold-pour joint filling compounds is described. The importance of proper technique of preparing and placing the newer materials is emphasized.

ENTRAINED AIR SIMPLIFIES WINTER CURING.....47-34

Price 35 cents.

JOSEPH J. SHIDELER, HAROLD W. BREWER and WILBUR H. CHAMBERLIN—Feb. 1951, pp. 449-460 (V. 47)

The investigation described was undertaken to determine the winter protection required to protect air-entraining concrete from damage by freezing. Cylinders made with Types II and V cement and various percentages of calcium chloride were cured at temperatures ranging from 10 to 70 F and tested for strength at ages ranging from 3 to 180 days. The resistance of these concretes to accelerated freezing and thawing was compared. The results indicate that the amount of winter protection, as presently specified by the Bureau of Reclamation, can be reduced when air-entraining concrete is used.

PRECAST CONCRETE CONSTRUCTION IN CANADA.....47-35

Price 35 cents.

OTTO SAFIR—Feb. 1951, pp. 461-468 (V. 47)

Precast concrete construction techniques as applied to a warehouse, parking garage and retaining wall project are described. Beams and columns for the precast concrete frames of a warehouse were cast flat in multiple forms on the ground slab. Cold weather required special precautions in mixing, placing and curing the concrete. In a parking garage, precast elements were combined with cast-in-place members to form an essentially monolithic structure. A retaining wall 25-ft high is of precast counterforts and 8-in. wall planks. Principal features of the job were the complicated shape of the counterforts and the weight of the completed units.

FACTORS IN PRESTRESSED GIRDER DESIGN.....47-36

Price 35 cents.

M. FORNEROD—Feb. 1951, pp. 469-480 (V. 47)

Following a general description of the Walnut Lane Bridge the stages of loading of the prestressed concrete girders and the sequence of construction operations as they affected the design are discussed. The owner's stress limitations are listed and longitudinal bending in girders, transverse bending in stiffener diaphrams and shear and principal tensile stress in the girder are considered. Safety factors are considered for various loading conditions.

PRECAST CONCRETE OFFERS PROTECTION AGAINST ATOMIC BLAST.....47-37

Price 50 cents.

ARSHAM AMIRIKIAN—Mar. 1951, pp. 497-516 (V. 47)

Precast concrete is an excellent means for providing protection against atomic blast. The technique is suitable for new construction and is adaptable to existing structures by providing them with a protective shell. Readily-assembled framing elements can be prefabricated at regional plants and stored or stockpiled at various points for immediate use in an emergency.

For design purposes, in this paper, shelters are divided into three groups in relation to their proximity to a probable target. Suggested arrangements and main details of assembly of a number of types of precast framing suitable for each group are presented. A general discussion of the protection problem, the needed weapons data and design criteria are also given.

FALSE SET IN PORTLAND CEMENT.....47-38

Price 35 cents.

R. F. BLANKS and J. L. GILLILAND—Mar. 1951, pp. 517-532 (V. 47)

False set of cement causes difficulties in mixing and placing and even though the stiffening is eliminated by job conditions, extra mixing or the addition of corrective admixtures, undesirable effects on the hardened concrete remain. It adversely affects water requirement, strength, bond between aggregate and matrix, brittleness and cracking, resistance to freezing and thawing, and air-entraining characteristics.

Correctives in the manufacture of the cement are proper cooling of mills or the use of stable calcium sulfate.

DIRECT DESIGN OF T-BEAMS.....47-39

Price 35 cents.

HENRY J. COWAN—Mar. 1951, pp. 533-544 (V. 47)

In designing T-beams by usual methods some initial dimensions of the section must be assumed, either the complete dimensions—and the maximum concrete and steel stresses computed to ensure they are less than the allowable stress—or the effective depth only assumed—with tension area computed and the maximum compressive stress in the flange checked to find if compressive reinforcement is required. The author emphasizes that both methods are likely to produce uneconomical sections and take more time.

The direct design procedure proposed enables the dimensions of a T-beam to be calculated for any given set of conditions. Curves are set up for finding the value of j, and the solution of problems by normal methods and "direct design" are compared by solving six examples. An "equivalent flange" method is proposed for the balanced design of T-beams when the area of compression reinforcement is specified, and results show little difference between exact and approximate methods of design.

ALKALI-AGGREGATE EXPANSION CORRECTED WITH PORTLAND-SLAG CEMENT.....47-40

Price 35 cents.

FEDERICO BARONA DE LA O—Mar. 1951, pp. 545-552 (V. 47)

Rather than a pozzolan, granulated basic blast furnace slag described in ASTM Specification C205 should be considered a latent or potential cement with a high siliceous glass content that requires the presence of hydrated lime and gypsum to hydrate properly. To correct alkali-aggregate expansion, a high proportion of slag (about 50 to 60 percent) should be used, which does not reduce strength; rather than lower percentages (20 to 30) which can not be exceeded with pozzolans unless strength is sacrificed. The portland-blast furnace slag cement produced in Mexico, meets ASTM requirements, presents satisfactory strength, low heat of hydration and low alkali content. The corrective action was investigated using Pyrex glass as reactive aggregate, and $NaOH$ to increase the alkali content of the different cements and blends to the same high value (1.23 percent).

Under similar conditions, with the same high alkali content, much smaller expansions were obtained with slag blends than with straight portlands. It is not intended to compare or recommend the use of slag instead of pozzolans, or the use of portland-blast furnace slag cement instead of modified, low-heat, or sulfate resistant portland cement, since in each case, the availability, cost and special conditions will determine what to use. In many cases, slag can be used advantageously to replace 50 to 60 percent of portland clinker.

LEAN MASS CONCRETE USED FOR INTERIOR OF DAMS.....47-41

Price 35 cents.

BYRAM W. STEELE—Mar. 1951, pp. 553-560 (V. 47)

It is not necessary for concrete in the interior and exterior of a dam to have equal durability. By placing 4-bag concrete on exterior faces and lean mass concrete (2½-bag mix) in the interior, it is possible to achieve both durability and economy in gravity dam construction. Lean mass concrete is important in minimizing volume-change cracking; the ensuing economy is an additional feature. The use of interior and exterior mixes in Corps of Engineers dams is described.

ACI'S PLACE IN A BILLION DOLLAR INDUSTRY.....47-42

Price 35 cents.

FRANK H. JACKSON—Apr. 1951, pp. 581-588 (V. 47)

Retiring ACI President Jackson examines Institute activities in relation to the concrete construction industry. ACI has won for itself a high place among the national professional engineering societies, but that eminence cannot be held without continual vigilance and increased effort on the part of all Institute members. Three major phases of activity are examined—annual and regional meetings, publications and committee functions. Recommendations are presented for improving and increasing Institute activities.

BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-51).....47-43

Price 50 cents (in special covers).

REPORT OF COMMITTEE 318—Apr. 1951, pp. 589-652 (V. 47)

Supersedes 44-1 and 47-18.

This code covers the proper design and construction of buildings of reinforced concrete. It is written in such a form that it may be incorporated verbatim or adopted by reference in a general building code, and earlier editions of it have been widely used in this manner.

Among the subjects covered are: Quality of concrete; allowable stresses; mixing, placing, curing and cold weather protection of concrete; forms; cleaning, bending, placing, splicing and protection of reinforcement; construction joints; general design considerations; flexural computations; shear and diagonal tension; bond and anchorage; flat slabs; columns and walls; and footings. The quality and testing of materials used in the construction are covered by references to the appropriate ASTM standard specifications.

THIN CONCRETE TOPPING RESTORES OLD PAVEMENT.....47-44

Price 35 cents.

H. WALTER HUGHES—Apr. 1951, pp. 653-660 (V. 47)

Lack of information on bonding new concrete to old led to laboratory bonding tests on wet and dry surfaces. Favorable results led to experiments on topping concrete road slabs, which required special techniques in bonding, screeding, compacting and finishing. No sign of surface disintegration has been observed after 6 years on any of the sections. Results indicate that the use of low water-cement ratio, graded mixes and compaction gives ample bond for thin toppings of floors or badly scaled pavements in which the concrete is sound. Concrete pavements containing low durability aggregates can also be given durable wearing surfaces with thin high-grade toppings.

UNIT BUILDINGS CUT CONSTRUCTION COSTS.....47-45

Price 50 cents.

L. G. FARRANT and W. C. HARRY—May 1951, pp. 669-680 (V. 47)

Design of a parking garage, separating the building into many parts, resulted in a highly functional and flexible low cost structure. The authors recognize that the principle of unit buildings can be applied to any type structure. A flat slab readily adapts itself to this principle and in this design it was again proved that the methods of analysis recommended by ACI can be applied to a slab of unconventional nature to obtain adequate safe design.

PLAIN AND REINFORCED CONCRETE ARCHES.....47-46

Price 50 cents.

REPORT OF COMMITTEE 312—May 1951, pp. 681-692 (V. 47)

A general method for the design of arches taking into account all important factors such as dead and live loading and the effects of volume changes, abutment movements and arch deflections. Because of the nonlinear relation between loads and the bending moments caused by deflections, it is found necessary to use an ultimate strength method of design taking into consideration the total effect of critical loading conditions including possible overload. The method permits the proportioning of a rib for optimum stiffness under assumed conditions, thus providing the most economical and satisfactory design.

While it has been general practice to disregard the effect of arch rib deflections, it should be included in the analysis to provide a proper safety factor against buckling and failure due to bending.

NEW APPROACH TO INHIBITING ALKALI-AGGREGATE EXPANSION.....47-47

Price 50 cents.

W. J. McCOY and A. G CALDWELL—May 1951, pp. 693-708 (V. 47)

Investigators have studied for a decade the chemical reactions between high-alkali cement and siliceous mineral constituents of some aggregates toward deterioration of concrete. Papers on this subject indicate a consensus that there are just two possible remedial measures when reactive aggregates are used—low-alkali cement or substitution of a pozzolanic material for 20 to 30 percent of the portland cement.

Investigative work focused off the beaten path of pozzolans and lowering cement alkali content resulted in experimental data which indicate that small amounts of certain materials added to high-alkali cement have an inhibiting effect on expansion reaction. For example, it has been found that 1 percent or less of specific salts will reduce expansion more than 75 percent in Pyrex glass mortar bar tests. This reduction was substantiated by similar mortar bar tests using a small percentage of opal and quartz sand as aggregate.

Additional information indicates that small amounts of certain proteins (0.2 percent or less) added to the cement appear to have a greater inhibiting effect on the expansive reaction than is obtained by comparable air entrainment effected by the conventional air-entraining agents. Such inhibitors appear to have no appreciable detrimental effect on the properties of the high-alkali cements as determined by ASTM specification tests for cement.

RECOMMENDED PRACTICE FOR THE APPLICATION OF MORTAR BY PNEUMATIC PRESSURE (ACI 805-51).....47-48

Price 50 cents (in special covers).

REPORT OF COMMITTEE 805—May 1951, pp. 709-720 (V. 47)

Supersedes 47-12.

This ACI Standard presents briefly the advantages and disadvantages of pneumatically-placed mortar and establishes recommended practices for placing and mixing shotcrete, qualifications and duties of workmen, preparation of surface before shotcreting, reinforcing, sequence of application, and other items involved in good shotcreting.

SPECIFICATIONS FOR CONCRETE PAVEMENTS AND BASES (ACI 617-51).....47-49

Price 50 cents (in special covers).

REPORT OF COMMITTEE 617—May 1951, pp. 721-744 (V. 47)

Supersedes 41-27 and 47-6.

Standard specifications for the construction of portland cement concrete pavement and base under normal conditions, including preparation of the subgrade.

The subjects covered include: materials; proportions of materials based on design for minimum strength or based on uniform cement factor; measurement and handling of materials; mixing; high-early strength concrete; subgrade preparation; forms; installation of joints and reinforcement; placing and finishing concrete; and curing.

DISCUSSION

Discussion closed January 1, 1951

Sept. Jl. '50

Some Observations on the Use of Reinforcing Steel in Concrete Pavements—Bengt F. Friberg

Tests of Paper Molds for Concrete Cylinders—Robert A. Burmeister

Admixtures in Concrete—W. T. Moran, F. H. Jackson, Bruce E. Foster and T. C. Powers

Improved Sonic Apparatus for Determining the Dynamic Modulus of Concrete Specimens—C. E. Goodell

Analysis of Three-Dimensional Beam-and-Girder Framing—Phil M. Ferguson

Discussion closed February 1, 1951

Oct. Jl. '50

Proposed Revision of Specifications for Concrete Pavements and Bases—Committee 617

Linear Traverse Technique for Measurement of Air in Hardened Concrete—L. S. Brown and C. U. Pierson

Influence of the Quality of Mortar and Concrete upon Corrosion of Reinforcement—Rachel Friedland

Proper Sand Grading Improves Mass Concrete—Thomas B. Kennedy

Water-Solubility of Alkalies in Portland Cement—J. L. Gilliland and T. R. Bartley

New Prestressing Method Utilizes Vacuum Process—K. P. Billner

Discussion closed March 1, 1951

Nov. Jl. '50

Proposed Recommended Practice for the Application of Mortar by Pneumatic Pressure—Committee 805

Determining Optimum Cross Sections for Prestressed Concrete Girders—Fred Uziel

Steam Curing Protects Winter Concreting—C. O. Crane

Analysis of Continuous Circular Curved Beams—Becla Velutini

Shear Resistance of Tile-Concrete Floor Joists—J. Neils Thompson and Phil M. Ferguson

Solution of Difficult Structural Problems by Finite Differences—Alfred Parme

Discussion closed April 1, 1951

Dec. Jl. '50

Proposed Revision of Building Code Requirements for Reinforced Concrete—Committee 318

Curing Concrete Pavements with Membranes—C. C. Rhodes

Blade Changes Improve Tilting Mixer—Glenway Maxon

Prototype Prestressed Beam Justifies Walnut Lane Bridge Design—Gustave Magnel

Finishing and Curing: A Key to Durable Concrete Surfaces—Myron A. Swayze

Economy Through Better Control of Reinforcing Steel—F. Tessitor and P. Rosewarne

Discussion closed May 1, 1951

Jan. Jl. '51

Proposed Revision of Manual of Standard Practice for Detailing Reinforced Concrete Structures—Committee 315

Coarse-Ground Cement Makes More Durable Concrete—Harold W. Brewer and Richard W. Burrows

Simple Equipment Economically Explores Prestressing—Marvin L. Mass and Jack R. Janney

Laboratory Tests of Spaced and Tied Reinforcing Bars—William T. Walker

Finishing Air-Entraining Concrete Pavements—Charles W. Allen

Effects of Lateral Loads on Arches—James P. Michalos

Getting More for Our Concrete Dollar—I. E. Morris

Discussion closes June 1, 1951

Feb. Jl. '51

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Thin Concrete Topping Restores Old Pavement—H. Walter Hughes

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New Approach to Inhibiting Alkali-Aggregate Expansion—W. J. McCoy and A. G. Caldwell

Recommended Practice for the Application of Mortar by Pneumatic Pressure (ACI 805-51)—Committee 805

Specifications for Concrete Pavements and Bases (ACI 617-51)—Committee 617

Title No. 47-50

Are Prestressed Bridges Cheaper?*

By STEWART MITCHELL†

SYNOPSIS

To study the use of prestressed concrete for highway bridges a pedestrian bridge over Arroyo Seco in Los Angeles was designed and placed under contract. In the United States, principal savings are expected to be for labor rather than in reduction of steel and concrete. Cast-in-place and precast prestressed bridges applicable to California sites are described.

Lack of technical data, cost information and experience restrict the use of prestressing techniques at present. Further studies and better specifications are needed to promote the use of prestressed concrete.

INTRODUCTION

In America, prestressed concrete seems to have been one of the most talked-about, and least-used, forms of construction. For more than 50 years it has been the subject of countless technical articles and discussions. About 15 years ago prestressed concrete became fairly popular in Europe and has since been used in many structures there. Highway engineers in this country are interested in knowing whether prestressed concrete has potential advantages in flexural bridge members or is a method suited only to European conditions.

Perhaps the most publicized prestressed bridge in America is the Walnut Lane structure in Philadelphia. This bridge, using I-shaped, prestressed concrete girders, has a span length of about 160 ft. A system of prestressing with cables has been used in a bridge built in Madison County, Tennessee.‡ Rapid erection and low cost are claimed for this type of construction (largely precast but partly cast-in-place) which seems particularly suited to short and moderate spans.

CALIFORNIA STUDIES AND EXPERIENCE

For several years the Bridge Department of the California Division of Highways watched with interest developments in prestressed concrete construction and collected an extensive bibliography on both prestressed and precast concrete.§ Then a suitable project was sought for experimenting with this type of structure.

*Presented at the ACI 47th annual convention, San Francisco, Feb. 20, 1951. Title No. 47-50 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 10, June 1951, *Proceedings* V. 47. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1951. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Principal Bridge Engineer—Planning, California Division of Highways, Sacramento.

‡*Engineering News-Record*, V. 146, No. 3, Jan. 18, 1951.

§Highway Research Board, Jan. 1949.

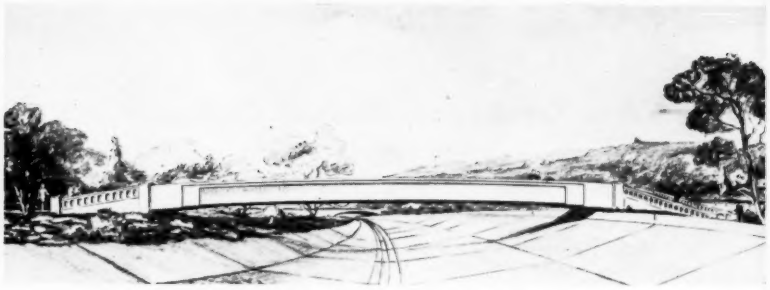

Fig. 1—Arroyo Seco pedestrian bridge

A pedestrian bridge was chosen for the initial venture into prestressing primarily because it is a safe and convenient type with which to experiment. Tests can be made without holding up vehicular traffic and there is little possibility of it being subjected to unregulated loads that greatly exceed the design live load such as can occur in a vehicular structure. However, the primary objective was to observe the practical results of the operations involved in prestressing and erection.

On September 21, 1950, bids were received for a prestressed concrete pedestrian bridge 110 ft long over the Arroyo Seco in Los Angeles, on a road leading to a state park (Fig. 1). A conventional reinforced concrete bridge of approximately the same span had previously been designed for this location so the two designs could be compared (Fig. 2 and 3). The prestressed design consists essentially of two concrete T-shaped girders supporting a conventional floor slab between them. Fig. 4 and 5 show the concrete stresses in the structure.

The design of this structure and the method of prestressing has been described previously by Hollister.* No attempt was made to achieve the ultimate economy of material which could have resulted in a smaller concrete section with less dead load and, therefore, less steel. It was considered more practical to experiment with a structure having a cross section suited to present day conventional construction. The general design involved no new

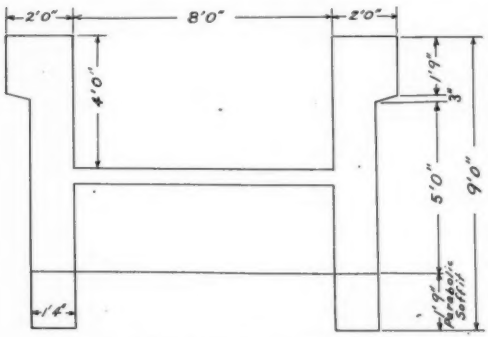

Fig. 2—Typical section, standard design

*Hollister, Leonard C., "Prestressed Concrete Design for Arroyo Seco Pedestrian Bridge," presented at meeting of Structural Engineers Assn. of California, Coronado, Oct., 1950.

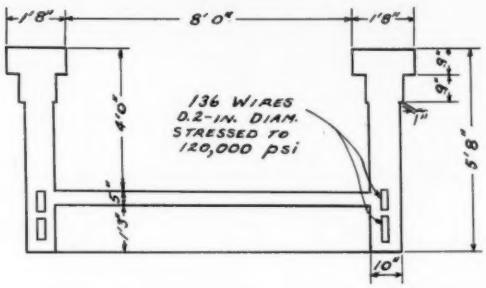

Fig. 3—Typical section, prestressed design

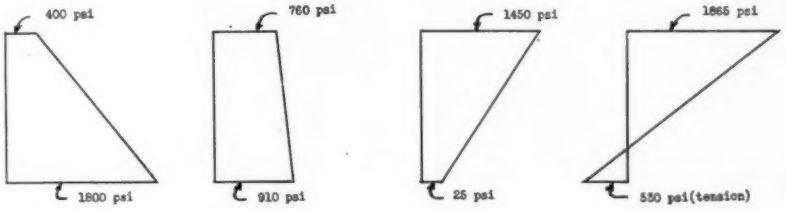
400 psi
1800 psi
Prestress plus girder DL (at time of prestressing)
760 psi
910 psi
Prestress plus girder DL plus sidewalk DL (after 15 percent loss in prestressing)
1450 psi
25 psi
Prestress plus girder DL plus sidewalk DL plus sidewalk LL (after 15 percent loss in prestressing)
1865 psi
530 psi (tension)
Prestress plus girder DL plus sidewalk DL plus 2 sidewalk LL (after 15 percent loss in prestressing)

Fig. 4—Concrete stresses at midspan

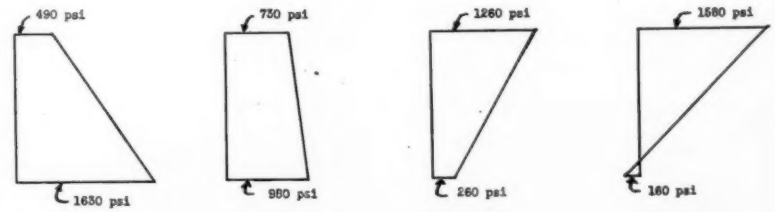
490 psi
1630 psi
Prestress plus girder DL (at time of prestressing)
730 psi
980 psi
Prestress plus girder DL plus sidewalk DL (after 15 percent loss in prestressing)
1260 psi
260 psi
Prestress plus girder DL plus sidewalk DL plus sidewalk LL (after 15 percent loss in prestressing)
1580 psi
160 psi
Prestress plus girder DL plus sidewalk DL plus 2 sidewalk LL (after 15 percent loss in prestressing)

Fig. 5—Concrete stresses at quarter-point of span

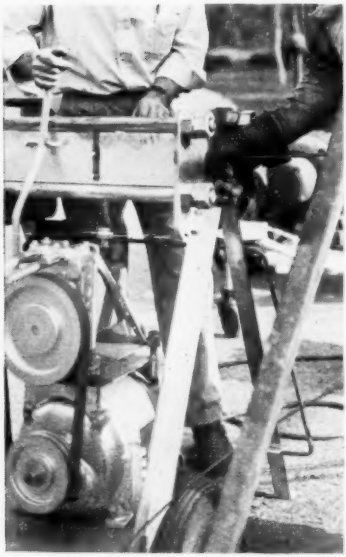

Fig. 6—Portable machine for forming button heads on wires

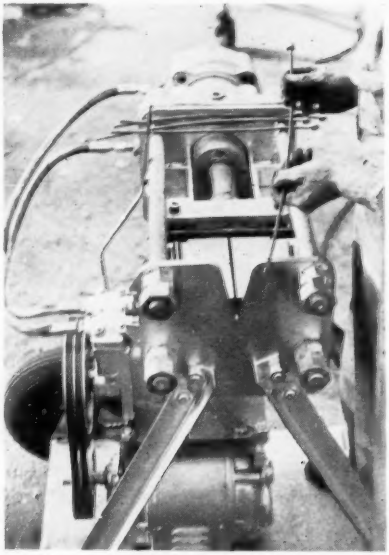

Fig. 7—Top view of button heading machine

departure from structures previously built in Europe. A two-span pedestrian overcrossing of similar dimensions designed to be erected in the same manner was built in France in 1942-43 and another of four spans in 1947-48.*

The prestressing contractor devised a new system of applying the prestressing to the high-tensile steel wires. A portable machine formed button heads on each wire before installation in the form (Fig. 6 and 7). The headed wires were threaded through a bearing plate (Fig. 8) at one end of the girder and a stressing block (holding the button heads) at the other and then put

Bulletin No. 10, International Assn. of Bridge and Structural Engineers, Dec. 1, 1950.

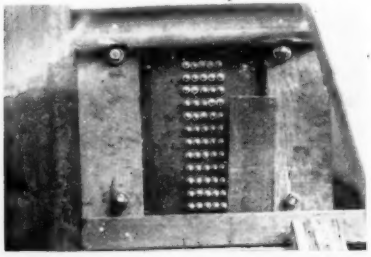

Fig. 8—Bearing plate and heads in place. Beam prestressed from opposite end

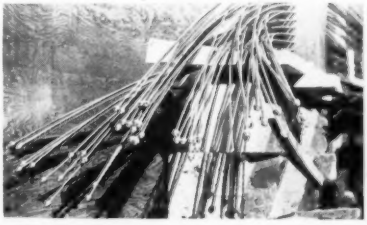

Fig. 9—Headed wires with washers and button heads for anchorage

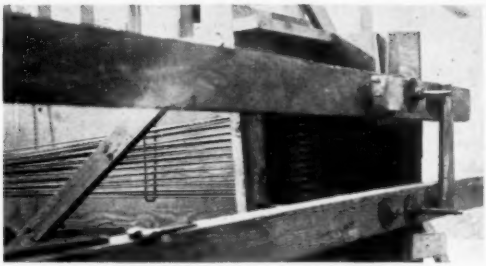

Fig. 10—Bearing plate with arch blocks in place

in their proper position in the form (Fig. 9). Arch blocks between the bearing plate and button heads assured proper position of the wires (Fig. 10). Cable groups were then enclosed in metal ducts (Fig. 11).

In cooperation with the Institute of Transportation and Traffic Engineering, University of California, some tests are being made on this structure. Carlson strain gages (Fig. 12) were embedded in the concrete girders to permit measurement of the actual stresses in the concrete while the steel wire was tensioned, and to measure the loss of prestress due to shrinkage and plastic flow. Measurements of stresses and deflections resulting from applied live loads are also contemplated.

With other reinforcement in place, the forms were closed and concrete was placed by bucket and vibrated in place (Fig. 13). When the concrete had attained sufficient strength the forms were removed and the stressing of the wires began. A jack (Fig. 14) engaged the stressing block and after the desired tension was attained, metal spacers (Fig. 15) were inserted to maintain the stress and the jack was removed and placed on the next block. The stressed girders are shown in Fig. 16. After the cables were grouted (Fig. 17) the girders were moved into place by four cranes (Fig. 18), and the floor slab formed and placed to complete the bridge.

RELATIVE COSTS

It is essential in a study of prestressed concrete, or any other design, to keep in mind the basic economic facts regarding concrete construction costs

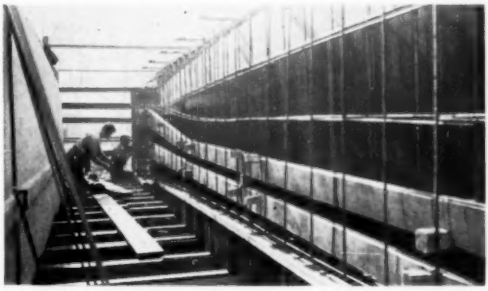

Fig. 11—Interior of girder form before closure. Metal ducts enclose prestressing wires

Fig. 12—Carlson strain gages and leads installed in forms

Fig. 13—Placing concrete for first girder

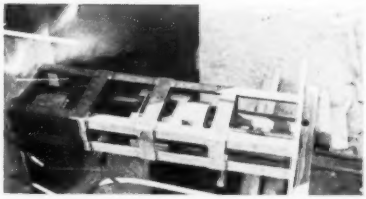

Fig. 14—Prestressing jack in place

in the United States as compared to those in Europe where prestressing is in more general use. Here, concrete materials and reinforcing steel are relatively cheap. Labor costs have always been relatively high in the United States but in recent years they have risen sharply in relation to the general cost of materials. Our salvation has always been the development of labor-saving methods and machinery. Therefore, to achieve economy, the theoretical possibilities of a new type of construction must lead to improved production methods, which in turn, will bring about greater use of the design and, consequently, lower cost.

In a strict sense, an economic comparison of different designs should take into account such factors as relative overload capacities, service life and salvage value, if any. The overload capacity of bridges now is anything but uniform. The ratio of resisting moment of a beam at working stress to that at ultimate stress is roughly the same for prestressed and conventional beams—probably somewhat in favor of the former. This is the figure usually quoted as the "safety factor." For the Walnut Lane girder* it is 2.69 whereas, due to the heavy dead load on a span of this length, the live load safety factor or overload capacity is about 10. The effect of cracking on the service life of concrete structures is a moot question but seems to be of importance

*Magnel, Gustave, "Prototype Prestressed Beam Justifies Walnut Lane Bridge Design," ACI JOURNAL, Dec. 1950, *Proc.* V. 47, p. 301.

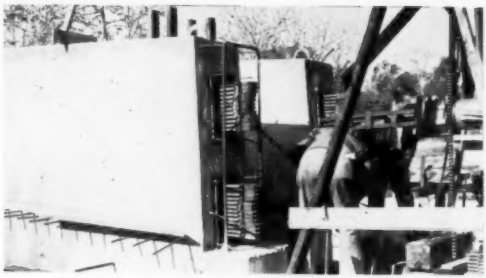

Fig. 15—Cables anchored on left girder. Stressing jack in operation on right

only under certain conditions. Likewise, salvage is not a factor of general importance in reinforced concrete construction. The prime consideration at present is the cost of fabricating and erecting structures of comparable design standard, and how they can be used to reduce the costs of appurtenant works. The probable costs and potential uses of prestressed concrete are considered from this standpoint.

A British viewpoint is expressed by the following quotation from an editorial in *Concrete and Constructional Engineering*, London, March, 1949, commenting on a conference of the Institution of Civil Engineers:

"Although the number of structures and civil engineering works constructed in this country wholly or in part in prestressed concrete is insignificant compared with those in plain and reinforced concrete, the types of structures now being erected may be an indication of the direction in which prestressed concrete may eventually become established. It has, however, yet to be established whether prestressed concrete is likely to be extensively used in Britain as an inherently good and economical method of construction, or whether it is in many cases an expedient the growth of which has been fostered mainly by the shortage of steel."

The type of structural design regarded as most economical in Europe is based on saving materials even at greater labor cost. Additional labor spent on formwork usually brings about some savings in materials, but in the United States, complicated formwork usually does not pay. The concrete I-beam for prestressed construction has found favor on the Continent since it offers a saving in material. However, it also requires relatively expensive formwork. Simple formwork results in greater salvage and re-use of expensive form material.

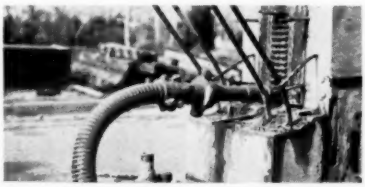

Fig. 16—Connection for pressure grouting of lower cables

Fig. 17—Girders after stripping of forms and tensioning of wires

Fig. 18—First girder being moved to final position

Particularly when concrete is poured in place, the slab has come to be used for conventional reinforced concrete spans which are longer than previously considered to be economical. It requires the minimum area of forms, minimum man-hours of labor to assemble, and provides maximum re-use of forms. Furthermore, a slab provides the least depth of superstructure where head room is important. A recent design for prestressed, precast construction uses box sections so connected that advantage is taken of slab action.

Comparative economic studies were made of the prestressed solid slab bridge, the prestressed hollow slab, and modifications of these two basic types. Designs, conforming to limitations imposed by AASHO specifications for highway bridges, indicate that prestressed slabs offer possibilities for economy in 60- to 120-ft spans. Structural types currently used for such spans include the concrete box girder, concrete T-beam, rolled steel beams and steel girders. Of course, factors other than strict construction economy also govern the choice of a type; these include appearance, clearances and provision for utilities.

In California freeway construction, solid concrete slabs of conventional design have been used frequently for spans up to 55 or 60 ft. For spans longer than 50 to 60 ft—up to about 140 ft—hollow cellular concrete box or steel girder spans are used extensively. By prestressing concrete slabs, their economical span length is increased from the present limit of 55 or 60 ft to about 80 ft. By incorporating hollow spaces in the slab (which might be formed by fiber tubes or boxes), the economical span length of the prestressed slab type might be increased to about 120 ft.

It seems possible that a prestressed slab might be built for as much as a dollar per sq ft of deck area less than the conventional type. However, this figure is based on the assumption that, ultimately, the cost of prestressing operations and materials will come down. It further assumes a comparatively low price for the necessary high strength concrete. A concrete designed

TABLE 1—COST ANALYSIS OF ARROYO SECO PEDESTRIAN BRIDGE

	Remove concrete	Excavation and backfill	Cl "A" concrete	Cl "E" concrete	Concrete rail	Cl "A" concrete, curb and sidewalk	Bar steel	Misc. steel	Prestress and reinf.		
Girders				50 cu yd $6000.00			5370 lb $ 590.70		5458 lb $5000.00	11590.70	
Slab			19 cu yd $1235.00				3270 lb $ 359.70			$1594.70	
Bearings								920 lb $ 322.00		$ 322.00	13507.40
Abut. No. 1		107 cuyd $ 428.00	38 cu yd $2470.00		30 lin ft $ 300.00		2750 lb $ 302.50	80 lb $ 28.00		$3528.50	
Abut. No. 2		120 cuyd $ 480.00	13 cu yd $ 845.00				1280 lb $ 140.80			$1465.80	$4994.30
Ramp 1		39 cu yd $ 156.00	24 cu yd $1560.00		90 lin ft $ 900.00		2830 lb $ 311.30			$2927.30	
Ramp 2		34 cu yd $ 136.00	16 cu yd $1040.00		50 lin ft $ 500.00		1500 lb $ 165.00			$1841.00	$4768.30
Misc. work	5 cu yd $100.00					8 cu yd $ 400.00					$ 500.00
Totals	$100.00	$1200.00	$7150.00	$5000.00	$1700.00	$ 400.00	$1870.00	$ 350.00	$5000.00		23770.00

to produce 5000 psi test cylinders is not always easy to obtain and usually requires extra cement to avoid a harsh mix. However, slab construction permits the placement of a stiffer mix with less cement, which is another point in its favor.

The contract price for the Arroyo Seco bridge is about the same as was estimated for a conventional reinforced concrete structure. The approximate quantity of concrete and reinforcement is:

	Standard design	*Prestressed design*
Concrete	88 cu yd	50 cu yd
Reinforcing steel	31,000 lb	5300 lb reinforcing bars 4700 lb wire
		10,000 lb

Assuming the cost of falsework, forms and fabrication to be approximately the same for either design, the saving in cost of furnishing and placing 38 cu yd of concrete at $12 per cu yd would be $460. At 11 cents per lb, the reduction of 25,700 lb of ordinary reinforcing steel would be $2830. The total of the two items is about $3300. Against this must be charged the cost of the prestressing wire and operations for which the contractor bid $5000 (Table 1).

However, a standard design would have required somewhat larger abutments and ramps. Also, being able to build the lighter prestressed girders on the ground and hoist them into place had an advantage over placing the concrete on falsework with resulting risk and delay from possible floods. Fabricating on the ground required only one form thus saving close to $3000. The saving in falsework is estimated to be another $2000. Against this must be charged about $2000 for handling the precast beams, leaving a net saving of $3000.

The contract price for furnishing high-tensile wire and performing all prestressing operations is nearly $1.00 per lb of wire. It is evident that the cost of labor and materials for anchoring, installing and grouting is relatively high. Factors other than the cost of prestressing being equal, it appears that the prestressed beam will show little, if any, saving in cost over ordinary types of construction until the prestressing cost is materially reduced. Naturally, this will be the ultimate objective of the suppliers of prestressing materials and services. However, when depth of superstructure, lighter precast members, or other such factors enter the picture, the prestressed girder seems to have a definite advantage.

OUTLOOK FOR PRESTRESSED CONCRETE DESIGN

Using about three miles of the Hollywood Freeway as an example it is found that the cost of structures is about two-thirds and grading one-third of the total cost of construction. The cost of structures runs from $1,500,000 to $2,000,000 per mile. It becomes imperative, therefore, to try and find ways in which this large cost of structure can be lowered.

Reducing bridge superstructure thickness is one possible method of achieving appreciable savings. In densely populated metropolitan areas it is not practicable to alter the grade of existing streets or railroads without causing excessive property damage. Consequently, freeways must be either depressed or elevated to provide necessary grade separations. It is apparent that any decrease in depth of the structures saves substantial quantities of excavation and embankment. Significant savings could also be realized in the cost of retaining walls. Ground water and drainage is a serious problem for depressed highways and it is evident that any reduction in depth of the depressed section would decrease cost of pumping, drainage and maintenance of pavement. Furthermore, reductions in the depth of excavated sections may, on occasion, require less width of right of way, permit the lengths of on or off ramps to be decreased, and reduce the cost of relocating underground utilities.

On California's state highway system, there are 5000 timber stringer spans. Sooner or later each of these spans will have to be replaced with more permanent construction. Many of these structures are in desert regions where labor, water, falsework timbers, form lumber and aggregates must be imported from a considerable distance. Cost of traffic control or maintaining detours is high so time is important and some method of reconstruction under traffic is usually desirable. Prefabricated construction has distinct possibilities under these conditions and already prefabricated bridge elements of all-welded steel and precast concrete have been used with satisfactory results. Under these conditions the lighter prestressed, *prefabricated* concrete units should have several advantages over ordinary reinforced concrete.

The smaller cross section and weight of prestressed deck units cuts down transportation costs and makes it possible to handle longer or wider units with a truck crane. Lightweight concrete would reduce the weight still more.

However, unless the cost of end-anchorages and prestressing can be reduced it is possible that pre-tensioned units might be cheaper for some conditions.

The real problem is not in finding a workable design and erection procedure as much as it is making the process commercially attractive. First of all there should be standard lengths, widths, skews and bearing details for the precast, prestressed bridge units that will meet the needs of state and county highway bridges, then development of "mass production" or "assembly line" methods and a good job of selling the product.

A West Coast concrete products company has developed a lightweight prestressed bridge plank for use in building construction using ordinary concrete blocks with planed bearing surfaces. The blocks are placed together and held by prestressed reinforcement. This type of construction could be developed for spans up to about 40 ft which could meet a large part of the needs of state and county highways. (On the California state highway system about 75 percent of all bridge spans are 40 ft or less.)

Saving concrete and steel is important under the defense program. It has been demonstrated that prestressed construction saves as much as 50 percent in the quantity of concrete and 80 percent in the weight of steel; but in the latter case the difference in quality must be considered. Prestressed concrete designs may be warranted even where there is no over-all monetary saving over conventional reinforced concrete design.

Difficulties have been encountered in obtaining needed information from companies interested in prestressed construction. Before advertising for bids on the Arroyo Seco structure, the California Bridge Department corresponded with prospective suppliers of prestressing materials or services. Plans and specifications were so prepared that any type, size or shape of wire or anchorage device that met the physical requirements would be acceptable. Organizations contacted either did not bid, or submitted a price to the general contractors that was about twice the engineer's estimate for the prestressing. The successful subcontractor, having learned of the project, submitted a bid which was within the engineer's estimate.

Our experience indicates it is difficult to obtain either firm estimates of cost for prestressed concrete construction or reasonably competitive bids. Until this is possible, the engineer will feel he must prepare competitive conventional designs to protect himself. If progress is to be made in any new type of construction, it is necessary that the various interested organizations jointly sponsor design and construction specifications. Information in convenient form is needed regarding size, availability and price of various types of high-tensile wire and prestressing equipment.

When a conventional concrete bridge is designed, the engineer is confronted with few difficulties involving proprietary materials. When there are patented features the designer must be careful to see that alternative methods are provided, particularly when he is employed by a public agency. In the case of prestressed construction, the patent situation is currently indefinite. Although any possible patents on the basic principles of prestressing expired

many years ago, it is believed the various commonly used anchorage devices are all still subject to patent control. Consequently, the engineer for a public agency must make provision in his design for the use of alternative methods of prestressing.

Another current difficulty is the lack of generally accepted design specifications. Since prestressed concrete acts differently from ordinary reinforced concrete, it is not always proper to use the same allowable stresses or limiting provisions. From habit, the American designer is hesitant to use stresses as high as those often used by foreign engineers. (More than 2000 psi for concrete or upwards of 120,000 psi for steel are unfamiliar figures.) Some published specification to rely upon—say, a tentative prestressed concrete design code—is urgently needed. Engineers of the Division of Highways found that writing one's own prestressed concrete specifications involved an enormous amount of research. They are still somewhat dubious that what they did write is the best that can be written.

SUMMARY

It is believed that the use of prestressed concrete for bridge construction in the United States will be accelerated when American engineers take their eyes off European practices and develop fabrication and erection methods suited to American conditions. Prestressed slab, and prestressed hollow slab type of construction seem to offer particular advantages for urban expressway and freeway separations or other conditions where shallow depths are pertinent. The use of prestressing will increase economical span lengths of concrete slabs and girders, and should make these types of construction competitive with concrete arches and steel superstructures.

Prestressed, prefabricated bridge elements also have possibilities for short-span bridge superstructures. Progress in this respect depends upon development of mass production facilities and methods, with as much standardization of span lengths, roadway widths and loading requirements as is possible under present conditions where highway bridges must fit a multitude of geometric designs and physical conditions.

Engineers must work with the fabricators and manufacturers to prepare design and contract specifications that will be generally acceptable.

Finally, it is hoped there will soon be more dissemination of working information regarding the details of prestressing equipment and firm quotations regarding prices. Then, and only then, can engineers reach intelligent conclusions regarding the advisability of using prestressing on specific projects.

ACKNOWLEDGMENT

Walter Kaucher of Los Angeles is the general contractor and Prestressed Concrete Co., Kansas City, Mo., has the subcontract for furnishing high-tensile steel wire reinforcement and performing all prestressing operations on the Arroyo Seco bridge. The bridge was designed by the Bridge Department of the California Division of Highways.

Discussion of a paper by Stewart Mitchell:

Are Prestressed Bridges Cheaper?*

By J. R. LIBBY and AUTHOR

By J. R. LIBBY†

The author has analyzed the economic potentialities of prestressed concrete for use in this country in the construction of bridges and, particularly, freeway structures from a very interesting standpoint. His remarks concerning prestressed prefabricated bridges were also very interesting. If such prefabricated structures could be mass produced, they should be of great value in replacing, for example, the many obsolete county highway bridges.

It should be pointed out that the amount of high tensile steel used in the prestressing of the Arroyo Seco bridge, as indicated in Fig. 3, does not agree with that shown in the quantity breakdown in the middle of p. 769. The data in Fig. 3 indicate that the total amount of steel required for prestressing both girders would be about 3200 lb when based upon a wire length of 110 ft. The quantity breakdown on p. 769 indicates that 4700 lb of wire was used in the structure. The prestressing units used in each girder of the subject structure, as described in the article "Simple Prestressing System for Pedestrian Bridge," *Engineering News-Record*, May 10, 1951, consisted of 25 units of five 0.25-in. diameter wires. The weight of wire required for prestressing the structure based on 125 wires of 0.25-in. diameter per girder is approximately 4600 pounds which agrees quite well with that shown on p. 769. The writer presumes the data shown in Fig. 3 were based upon preliminary design drawings.

In reviewing the author's suggested figure of $1.00 per lb of wire for use in estimating the cost of prestressing bridge structures, the writer feels this cost would not be applicable to all methods of prestressing. Since the author has indicated his conclusions that the cost of labor and materials for anchoring, installing and grouting is relatively high, it seems evident to the writer that one way to reduce the cost of prestressing would be to simplify and reduce these operations. To reduce labor and material required for accomplishing the prestress, the writer believes that either galvanized bridge strand could be used to advantage for the prestressing elements, or that a larger number of wires should be prestressed at one time.

Based upon the prestressing force of 715,000 lb required in each girder of

*ACI JOURNAL, June 1951, *Proc.* V. 47, p. 761. Disc. 47-50 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 4, Dec. 1951, *Proceedings* V. 47.

†Structural Engineer, Framed Structures Division, Structures Research Dept., U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, Calif.

the Arroyo Seco structure, as described in the *Engineering News-Record* article mentioned above, the prestressing force could be obtained either by tensioning ten 1-in. galvanized bridge strands two at a time, or by stressing 25 wires at a time rather than five wires. There would be a reduction in field labor costs by doing this since the field tensioning operations could be reduced from 25 to five operations per girder. The writer believes that little, if any more, work would be required to cut 25 wires to the same length for stressing than would be required for 25 wires in groups of five wires. Also, it is questionable if any individual wire would be stressed higher than average in a 25 wire cable than in a five-wire cable.

The equipment required for prestressing the cable units is not expensive and is readily available today. By using a 10,000 psi hydraulic pressure system for prestressing, the required jack could be kept to a convenient size so that it could be handled easily in the field. Hydraulic pumping units that have capacities of 10,000 psi pressures are available in Los Angeles today for use with either compressed air or electric power operation. The writer believes the equipment necessary for prestressing the bridge with cables, as described above, including the pump, jack, hose, pressure gages, valves, and necessary jigs could be obtained in Los Angeles for less than $1000.00.

Because this indicates a lower cost for prestressing with large cables than for prestressing by the means described by the author due to decreased labor costs, the writer is curious to learn if these methods of prestressing with large cables were considered in the design of this beam, and if any bids for prestressing based on these methods were received.

AUTHOR'S CLOSURE

The writer appreciates receiving Mr. Libby's comments on his paper. As Mr. Libby infers, the relative cost comparisons on p. 769 and 770 were based on preliminary figures and those published later elsewhere are, of course, more accurate. The writer's primary purpose was to point out the approximate value of factors which may enter into a comparison of costs between prestressed and standard reinforced concrete structures.

As indicated in the paper, specifications for the pedestrian structure were left as wide open as possible for all approved methods of prestressing with the hope that proponents of all such methods would submit bids. However, it was understood that only two firms quoted prices to the general contractors who bid on the project and that neither firm made use of cable reinforcing. This situation was used by the writer to substantiate the apparent need for more competition if prestressed concrete is to compete with ordinary reinforced concrete design.

The Prestressed Concrete Corporation, subcontractor for the prestressing for this pedestrian overcrossing, undoubtedly could reduce the cost of their method on another similar job. However, Mr. Libby's cost comparisons are interesting even though other factors do enter into the total cost and bid prices are really the final criterion.

Title No. 47-51

Long-Time Study of Cement Performance in Concrete

Chapter 7. New York Test Road*

By F. H. JACKSON† and I. L. TYLER‡

SYNOPSIS

Test procedure, materials, mix proportions and construction procedures on the New York Test Road, a part of the Long-Time Study of Cement Performance in Concrete, are described. All 27 of the Long-Time Study cements were used in conjunction with a nontest cement for adjusting equipment and mix proportions. Performance of the test sections is assessed with respect to durability of the concrete as a *material*, the only property directly studied in relation to the cement used.

Seven and one-half years after the test pavement was completed no one cement proved superior to the others tested. However, the effects of air entrainment in improving resistance of the pavement to scaling and weathering overshadowed all other variables. The use of abrasives with ice control chemicals caused scaling on non-air-entraining concrete. A 2 to 3-in. increase in slump of the test concrete has had no appreciable effect on the durability of the concrete.

INTRODUCTION

The New York Test Road (Projects 1-1A-1B) is the largest of three major projects to study cement performance in pavement concrete; others are in South Carolina (Project 2) and Missouri (Project 3). The three locations provide a wide range of exposure conditions. Each project design and construction method was common to the locality—a normal range in accepted practices in different parts of the country. Aggregates were typical of those normally used in the three areas. The three test roads total 6 miles of two-lane pavement and constitute a major part of the Long-Time Study as to quantities of materials and costs of construction. Of the three projects only the New York road, with by far the most severe exposure, warrants the presentation of first significant test results.

*Received by the Institute Nov. 20, 1950. Title No. 47-51 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 10, June 1951, *Proceedings* V. 47. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1951. Address 18233 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Principal Engineer of Tests, Bureau of Public Roads, Washington, D. C.

‡Member American Concrete Institute, Manager, Field Research Section, Portland Cement Assn., Chicago, Ill.

LOCATION AND GENERAL FEATURES

The road is in Allegany County, State Route 17, between Wellsville and Bolivar where winter conditions are severe. Including interspersed non-test sections, it comprises 2½ miles of standard two-lane highway. There are 168 test sections of concrete pavement, 12 ft wide and 150 ft long. The test road was part of a 7-mile construction contract which included grading, drainage and pavement.

The test pavement is in three parts—Projects 1, 1A and 1B. Project 1, in which 21 untreated and three air-entraining cements were used, is comparable with Projects 2 and 3, the South Carolina and Missouri test roads. It is on a nearly straight section having only slight grades where it was anticipated that the use of chemicals for ice control would be unnecessary. It was constructed in accordance with normal concrete pavement practice in New York State.

Project 1A with three untreated and three air-entraining cements made up a relatively short length of pavement similar to that of Project 1. Concrete slump was increased from the 3 in. used in Project 1 to slightly less than 6 in. Project 1A demonstrates the effects of added mixing water and provides a further comparison of performance of air-entraining and non-air-entraining cements. Direct comparisons can be made between the performance of the cements of this project and the performance of the same six cements in Project 1 with the quantity of mixing water as the only difference.

In Project 1B, six untreated cements and their six air-entraining counterparts made from the same clinkers were used in concrete of the same mix proportions and slump as in Project 1. This project has grades up to 4.8 percent where ice control chemicals would ordinarily be used. The purpose was to show the effects of air entrainment on the performance of concrete pavement where chemically treated abrasives were frequently applied for ice control.

The test sections were constructed between August 15 and October 9, 1942. The pavement was placed over an existing roadway improved by several small changes in grade and a few in alignment. A 6-in. rolled run-of-bank gravel foundation course was placed over the old subgrade and on new cuts and fills. A rainy summer and fall kept the subgrade soft during much of

Fig. 7-1—General view, Project 1. Transverse cracks in right lane

Fig. 7-2—Cracks due to settlement

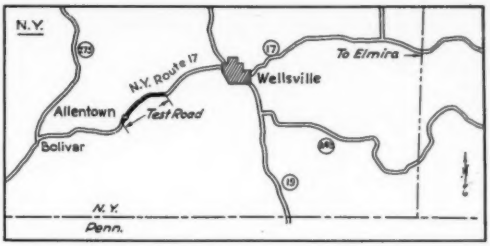

Fig. 7-3—Location of New York test road

the paving period and it was expected that unequal settlement and frost heaves might cause some structural cracking of the test slabs. (Fig. 7-1 and 7-2).

The location of the road with respect to towns and main travelled roads is shown in Fig. 7-3. Fig. 7-4 shows profile and dates each pavement test section was cast. Fig. 7-5, showing a plan and profile of Project 1B, is of particular interest since it is on this project that the developments of greatest significance are taking place.

TEST SECTIONS

On each project each cement was used in four test sections of equal length. The cements were used in rotation, constructing one 12 x 150 ft (single lane) section at a time. Thus there are four repetitions or *rounds* in which each cement appears in each project. By this procedure there was a considerable time interval between the construction of test sections in which any one cement appears. This was to avoid duplication of placing conditions, including

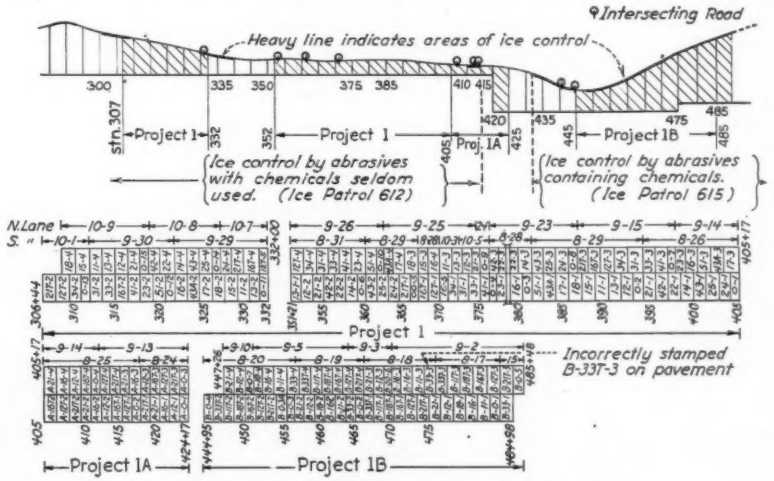

Fig. 7-4—Profile of test road showing dates of placing concrete

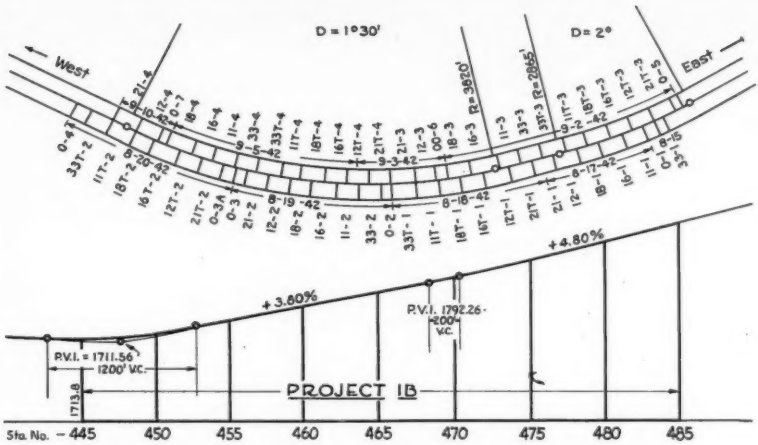

Fig. 7-5—Plan and profile of Project 1B showing location of test cements

weather, and subgrade conditions which might have occurred had all the test sections with one cement been constructed successively.

Expansion joints were at 75 ft intervals with premolded joint fillers and load transfer devices at each joint. There were no other joints or transverse planes of weakness. Fig. 7-6 shows the pavement cross section and Table 7-1 shows essential design details.

For practical reasons the change from one cement to another was normally made a short distance beyond the transverse joint marking the nominal boundary of the test section. As a result the different concretes do not necessarily have accurately marked boundaries, and the "over-run," concrete carried past the transverse joints, must be considered when the test sections are studied. With the exception of the last section placed each day which terminated at a transverse joint, the over-run is present at all section boundaries. However, each test section has at least one joint completely surrounded by concrete of the same test cement, and except for the last section placed each day there are two such joints in each test section. Should over-run ever become of critical importance, the construction records show the actual locations of changes in cements. In most cases the change in color of the pavement identifies the changes in the field.

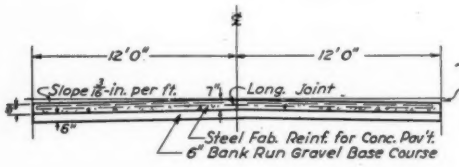

Fig. 7-6—Cross section of Projects 1, 1A and 1B

TABLE 7-1—SUMMARY OF CONSTRUCTION DATA

Feature	Description	Feature	Description
Paving started	Aug. 15, 1942	Paver	27E Foote (2)
Paving completed	Oct. 9, 1942	Mixing time	1¼ min.
No. of cements	Proj. 1-24; 1A-6; 1B-12	Power spreader	None
Test sections each cement	4	Transverse finisher	Lakewood
Pavement width	24 ft	Longitudinal float	Power (Koehring)
Pavement cross section	8-7-8	Compacting at forms and joints	Spading
Constructed	Half width (12 ft)	Straightedge	Wood (seldom used)
Length test section	150 ft	Final finish	Wood and burlap drags
Width test section	12 ft	Broomed	Yes
Expansion joints	75 ft intervals	Curing	Mats 72 hr.
Contraction joints	None	Nominal cement content	6½ bags per cu yd
Load transfer	At expansion joints	Slump	1 and 1B-3 in.; 1A-5 in.
Load transfer assembly	Spade type, premolded filler	Fine aggregate	Natural sand (N.Y. 47L Spec.)
Reinforcing	Welded mesh	Coarse aggregate	Crushed gravel (2 sizes)
Center joint	Formed with key, screwed tie bars	Auxiliary specimens	None (cored later)

MATERIALS

All 27 of the Long-Time Study cements were used. Complete data on physical and chemical properties of the cements, representing all five ASTM types, are given in Chapter 3, Chemical and Physical Tests of the Cements, of the Long-Time Study Report.* Table 7-2 shows calculated compound composition and other data on the cements. Additional test data on premature stiffening tendencies were obtained from laboratory tests after construction of the test road. These data (Table 7-3) are discussed in a later section on concrete placing. A "local" (nontest) cement was used in the first pavement section of each day's test construction and in all other nontest concrete.

The test cements were delivered in 6-ply asphalt lined paper bags except for Cement 33T which was packed in 4-ply bags. All cements were stored in a temporary warehouse for one to three months. Small portions of a few of the cements were slightly damaged by rain penetrating the temporary building, but the heavy bags prevented serious damage.

Three sizes of concrete aggregates were used: natural sand meeting the New York State 47L specification and crushed gravel in two sizes, No. 4 to 1-in. and 1-in. to 2-in. The sand came from Irving, N. Y., and the crushed gravel from Alfred Station. All aggregates were washed and were received wet at the batch plant. Table 7-4 shows typical sieve analyses for the three aggregates. Tables 7-5 to 7-7 inclusive show physical and chemical test results on the aggregates.

Mixing and curing water was pumped from streams adjacent to the right of way. Since the road was constructed in an oil producing area with some drilling in progress, there was some concern about quality of the water and several samples were analyzed for possible deleterious contaminations. The average chemical composition of water used in the three projects was as follows:

*ACI JOURNAL, Apr. 1948, *Proc.* V. 44, p. 745.

Constituent	*Amount*
CaO	87 ppm*
MgO	35 ppm*
SO_3	30 ppm*
Na_2O	441 ppm*
K_2O	9 ppm*
CO_2	10 ppm*
HCO_3	92 ppm*
Cl_2	687 ppm*
Total solids, 105 C	1295 ppm*
Solids after ignition	1142 ppm*
pH	8.5 (range 7.4 to 9.1)

*Parts per million

TABLE 7-2—CALCULATED COMPOUND COMPOSITION AND OTHER DATA ON CEMENTS*

Cement No.	Calculated compound composition,† percent by weight					Meas. value,	Minor oxides, percent by weight				Specific surface,
	C_3S	C_2S	C_3A	C_4AF	$CaSO_4$	C_3A	Free *CaO*	*MgO*	Na_2O	K_2O	ASTM
Type I											
18	45.0	27.5	13.0	6.8	3.1	3.8	0.3	2.6	0.12	0.13	1750
12	45.0	28.0	12.6	7.3	2.6	5.6	0.1	3.1	0.28	0.40	1815
11	51.0	21.0	12.1	7.2	2.7	3.1	0.4	3.7	0.21	0.51	1820
15	65.5	9.0	12.0	7.5	3.3	8.6	0.4	0.8	0.08	0.23	1845
17	51.5	23.0	10.4	9.3	2.9	4.2	0.4	1.1	0.08	0.43	1770
13	50.5	26.0	10.0	6.5	2.8	1.6	1.6	1.1	0.04	0.19	1665
14	43.0	32.0	8.2	9.2	2.9	3.3	0.2	2.5	0.06	1.30	1880
16	53.0	21.5	7.4	10.7	2.9	4.4	0.7	2.1	0.23	0.46	1785
Type II											
22	41.5	33.0	6.6	11.7	2.4	2.2	0.1	3.2	0.24	0.37	1775
21	40.5	38.0	6.3	9.8	2.1	2.0	0.7	1.3	0.22	0.40	1630
24	41.5	28.5	5.4	14.8	3.0	2.9	0.9	3.1	0.06	1.30	1925
25	34.5	39.0	4.8	14.8	3.2	1.5	0.2	2.2	0.21	0.54	1825
23	51.5	22.5	3.8	16.6	2.6	1.2	0.4	0.9	0.59	0.14	1875
Type III											
31	56.5	16.0	10.7	6.4	3.8	3.2	1.5	3.3	0.23	0.22	2795
33	59.0	13.5	10.4	7.7	3.9	7.2	1.8	1.4	0.21	0.44	2530
34	64.0	10.5	5.6	10.0	2.9	4.2	2.3	2.5	0.28	0.28	2465
Types IV and V											
43	24.5	48.0	6.2	13.8	3.6	2.8	0.1	1.6	1.00	0.08	1965
43A	29.5	50.5	5.3	9.3	3.2	1.0	0.4	1.1	0.33	0.01	1915
41	20.0	51.0	4.7	15.2	3.4	1.5	0.4	3.0	0.06	1.19	1915
51	41.5	39.0	3.8	10.0	2.4	1.1	0.5	1.7	0.08	0.22	2025
42	27.0	55.0	3.4	8.2	2.6	0.9	0.2	1.8	0.16	0.26	1920
Air-entraining											
18T	45.0	28.5	13.2	6.6	2.9	3.8	0.4	2.6	0.14	0.13	1960
12T	45.0	28.0	12.5	7.1	2.6	5.6	0.1	3.1	0.31	0.40	1790
11T	51.0	21.5	12.2	7.1	2.6	3.1	0.5	3.7	0.22	0.51	2010
33T	56.5	16.5	10.4	7.4	3.7	7.2	1.8	1.5	0.21	0.46	2510
16T	51.5	22.5	7.9	10.4	3.1	4.4	0.8	2.0	0.23	0.44	1680
21T	38.5	40.0	6.6	9.8	2.2	2.0	0.6	1.3	0.21	0.43	1750

*Data from Tables 3-4 and 3-5, Chapter 3, ACI JOURNAL, Apr. 1948, *Proc.* V. 44, and Table 4-2, Chapter 4, ACI JOURNAL, May 1948, *Proc.* V. 44.

†Corrected for free *CaO*; not corrected for minor oxides. These values are correct and are based on calculations from oxide analyses expressed to two decimal places. Values in Table 3-4, Chapter 3, were substantially the same except for some minor errors which escaped detection.

TABLE 7-3—DATA ON PREMATURE STIFFENING TENDENCIES OF THE CEMENTS AND OBSERVED CHARACTERISTICS IN THE FIELD*

Cement No.	P-1, mm†	P-3, mm‡	B, mm§	Premature stiffening tendencies in field
43	1	4	16	Marked
43A	1	2	6	Marked
12	1	22	22	Moderate
16	1	35	21	Moderate
22	3	24	30	Moderate
15	5	30	7	Slight
17	6	40	36	Slight
23	17	35	10	Slight
18	12	26	14	Questionable
31	1	18	3	Questionable
11	34	34	29	None
13	34	36	33	None
14	35	39	38	None
21	31	35	28	None
24	36	38	39	None
25	40	40	39	None
33	22	38	38	None
34	37	38	32	None
41	34	39	40	None
42	35	33	27	None
51	29	39	26	None

*Penetration tests made in Vicat apparatus using pastes made of Long-Time Study Cements in storage at the Naperville Test Plot with 26 percent mixing water.
†P-1 = Penetration of ¼-in. needle; 15 min. after mixing, paste mixed 1 min.
‡P-3 = Penetration of ¼-in. needle; 15 min. after mixing, paste mixed 3 min.
§B = Penetration of ½-in. needle; 2½ min. after mixing, paste mixed 3 min.
Bureau of Reclamation Specification No. 898-D, Mar. 1937.

The contamination is not believed to be sufficient to affect performance of the concrete.

MIX PROPORTIONS

The concrete mix used corresponded closely with that established for pavement concrete in adjacent areas of New York State. In the calculated mix data (Table 7-8) there were some differences in the cement content among the different cements. These differences are mainly the result of the different weights of cement per bag, as discussed later. Small differences in mixing water and entrained air, batching errors, etc., also contributed to the deviations. Cement 41, not considered an air-entraining cement, produced appreciably more air in concrete than the other non-air-entraining cements and as a result the mix data show it to have a somewhat lower cement content than had been expected.

TABLE 7-4—TYPICAL SIEVE ANALYSIS OF AGGREGATES—PERCENT PASSING (SQUARE OPENINGS)

	No. 100	No. 50	No. 16	No. 4	¼	½	1	1⅞	2¼
Irving Sand	1	14	63	94	100				
Crushed gravel No. 1 and 2					9	43	94	100	
Crushed gravel No. 3A							5	92	100

TABLE 7-5—BULK SPECIFIC GRAVITY, SATURATED-SURFACE DRY, AND 24-HOUR ABSORPTION OF AGGREGATES (7 TESTS)

	Bulk specific gravity			Absorption, percent		
	Max.	Min.	Ave.	Max.	Min.	Ave.
Sand	2.56	2.53	2.54	2.01	1.70	1.9
Gravel, No. 1 and 2	2.58	2.56	2.57	1.69	1.52	1.6
Gravel, No. 3A	2.62	2.58	2.59	1.60	1.25	1.4

TABLE 7-6—RATIONAL ANALYSIS OF SAND (4 TESTS)—PERCENT

Test No.	Kaolin	Feldspar	Quartz	$CaCO_3$	$MgCO_3$	R_2O_3
16599	4.0	48.7	41.5	1.0	1.1	3.7
16602	4.4	49.3	40.0	1.7	1.1	3.4
16605	3.5	48.8	37.9	3.5	1.7	4.6
16618	3.2	22.1	43.8	21.0	6.2	3.7

TABLE 7-7—SODIUM SULFATE TESTS OF AGGREGATES (7 TESTS)—PERCENT WEIGHT LOSS

Material	Size range	5 cycles			10 cycles		
		Max.	Min.	Ave.	Max.	Min.	Ave.
Sand	Minus No. 4	12	3	7	23	3	9
Crushed gravel, No. 1 and 2	⅜ to No. 4	8	3	4	16	7	10
	¾ to ⅜ in.	35	2	16	42	14	26
	1½ to ¾ in.	32	7	20	58	26	38
Crushed gravel, No. 3A	1½ to ¾ in.	23	0	10	62	16	36
	2½ to 1½ in.	9	0	3	45	19	32

TABLE 7-8—FIELD MIX DATA

Average of all data—4 Rounds

Cement No.	Cement, sacks per cu yd	Net W/C, gal. per sack	Slump, in.	Unit weight, lb per cu ft	Approx. air,* percent
Project 1					
11	6.48	4.7	3.1	150.6	0.7
12	6.49	4.9	3.1	150.4	0.4
12T	6.44	4.7	3.5	147.6	2.8
13	6.59	4.7	2.8	150.2	1.0
14	6.55	4.7	3.0	150.6	0.7
15	6.49	4.9	2.7	149.7	1.0
16	6.49	4.8	3.2	150.4	0.7
16T	6.43	4.6	2.9	143.5	5.9
17	6.51	4.9	3.1	150.3	0.6
18	6.53	4.8	3.4	150.3	0.7
21	6.44	4.8	2.6	150.6	0.6
21T	6.44	4.7	2.8	147.2	3.1
22	6.50	4.9	3.1	150.4	0.5
23	6.63	4.8	3.0	150.5	0.4
24	6.51	4.6	3.4	150.5	0.9
25	6.68	4.5	2.6	150.5	0.9
31	6.52	5.1	3.1	149.2	0.9
33	6.02	5.3	3.2	149.0	1.4
34	6.34	5.1	3.3	149.3	1.2
41	6.39	4.5	3.1	149.3	2.1
42	6.55	4.8	3.0	150.8	0.4
43	6.55	4.8	2.8	150.5	0.5
43A	6.51	4.8	3.0	150.6	0.5
51	6.45†	4.8	3.2	150.6†	0.6†
Project 1A					
12	6.45	5.1	4.9	149.5	0.8
12T	6.38	5.1	5.3	147.0	2.7
16	6.46	5.2	5.1	150.1	0.2
16T	6.51	4.9	5.6	142.9	5.5
21	6.39	5.2	5.5	150.1	0.1
21T	6.34	5.2	5.3	146.4	3.0
Project 1B					
11	6.47	4.8	3.4	150.3	0.8
11T	6.34	4.7	3.3	142.9	6.2
12	6.48	4.9	2.5	150.0	0.7
12T	6.44	4.8	2.6	147.8	2.6
16	6.49	4.8	2.8	150.9	0.5
16T	6.58	4.7	3.3	145.6	4.1
18	6.53	4.8	2.9	150.1	0.8
18T	6.54	4.9	2.7	147.5	2.5
21	6.46	5.0	2.9	150.9	—
21T	6.45	4.7	3.1	148.1	2.5
33†	6.01	5.5	3.1	149.1	0.9
33T	6.40	5.0	3.4	145.6	4.1

*Calculated from unit weight.
†Average of 3 rounds.

For non-air-entraining cements the ratio of sand to total aggregate by absolute volume was 37 percent. For air-entraining cements the sand was reduced in absolute volume by an amount equal to the increase in air (above 0.8) due to entrainment of air and increased by an amount equal to the reduction in volume of the mixing water brought about by the effect of air entrainment. It was the consensus of competent observers that the mix proportions approached optimum values for the type of placing and finishing equipment used.

CONSTRUCTION PROCEDURE

Construction practices were those normally followed by the New York Department of Public Works in the Hornell District and there were no special requirements for the test construction other than for details having to do with uniformity of materials and operations. Each day's work began with placing one pavement section using the local (nontest) cement. This was followed immediately by test concrete. During placing of the nontest section mixing and placing operations were stabilized and mixing water requirements were established. Thus many of the variable conditions commonly present while paving is getting under way were confined to the nontest sections.

The sequence in which the cements were used was based upon bleeding characteristics observed in the laboratory. While field performance was not exactly the same as the laboratory tests indicated, the sequence chosen kept the cements of approximately similar bleeding characteristics together. Thus, adjacent test sections had somewhat similar finishing characteristics.

Construction was on a single shift basis on a schedule somewhat slower than anticipated. Subgrade preparation and paving operations were severely hampered by rainy weather which persisted throughout much of the eight-week construction period. The transverse finishing machine, which had seen years of service, was not sufficiently powerful to rapidly compact concrete of 2 to 3-in. slump. As a result it was necessary to use concrete of 3-in. slump or slightly more to maintain an acceptable construction schedule and to assure proper compaction of the concrete. As a further complication a few of the test cements tended to stiffen prematurely, a phenomenon not noticed on the other two test roads constructed the year before.

Undesirable effects of the tendency toward premature stiffening were partially overcome by shortening the time between mixing and compacting and finishing the concrete. Under extreme conditions slightly more than the normal amount of mixing water was used to avoid delays. Table 7-3 shows test data on premature stiffening obtained in the laboratory after completion of the road, as well as the extent to which this tendency was noted in the field.

Batching

The test cements were batched by bags at the cement warehouse. The local (nontest) cement was batched in bulk through a weigh batcher. In packing test cements in the heavy 6-ply waterproof bags it had been found

difficult to hold the contents to exactly 94 pounds, though in most cases the weights were reasonably accurate. In one case (Cement 33) the weight was found to be decidedly low. To determine the quantity of each cement actually used in the test concrete, a number of bags of each were weighed and the average weight for each cement established for calculating mix data.

Aggregates were batched by weight through a single cumulative weigh batcher. The weighing equipment was checked frequently and scales were adjusted to maintain the delivery of accurately weighed materials. Appropriate weight corrections were made for moisture content of each size of aggregate. Batcher operators were experienced and cooperative and it was believed that batching errors seldom exceeded 2 percent.

Mixing water was batched through the batching equipment of the pavers. As a check on the water batchers, 2-in. displacement-type water meters were installed in the water lines to the pavers' reserve tanks. With these meters the amount of water delivered to the paver during any period could be compared with the indicated amount of water batched into the concrete during the same interval. Each water batcher was calibrated before construction began and check calibrations were made during construction whenever the water meters indicated any change in the initial calibration.

Aggregates and cement were delivered to the pavers in batch trucks carrying two batches to a load. The haul was from 2 to a little more than 5 miles for aggregates and approximately ½ mile less for the test cements. The batch compartments were maintained in satisfactory condition and leakage from the trucks or spillage from one compartment to another were not serious problems.

Control

With the 27 test cements and the local cement used in the test road, batching and delivering concrete materials to the paver in proper sequence and in proper quantities without confusion was a problem. With good cooperation between batch plant operators, truck drivers and the concrete mixing and placing personnel, it was possible to deliver batches containing the different cements so as to keep the cements in their proper locations in the test sections without confusion or delays to paving.

The quantity of mixing water was controlled by the consistency of the concrete needed to secure satisfactory operation of placing and finishing equipment for adequate consolidation of the concrete. Slump was used as the measure of consistency and slump tests of the concrete as deposited on the subgrade were made frequently, providing direct control. An exceptionally large number of moisture determinations on aggregates (some 1500 in all) correlated with the slump tests furnished information from which dependable data on water-cement ratios could be calculated. Average water ratios for each cement in each of the three projects are shown in Table 7-8.

Average slumps for each round, for each cement, are shown in Table 7-9. Variability of the slumps is indicated by the standard deviations. The averages include slump tests made when it appeared that the concrete mix was

TABLE 7-9—NEW YORK TEST ROAD—SLUMP OF FRESH CONCRETE, IN INCHES, BY ROUNDS

Cement	Round 1			Round 2			Round 3			Round 4		
	n*	Ave.	σ†	n*	Ave.	σ†	n*	Ave.	σ†	n*	Ave.	σ†
Project 1												
11	8	2.6	0.8	9	2.7	0.6	11	3.8	1.0	8	3.1	0.7
12	10	3.2	0.9	8	2.8	0.6	10	3.4	1.1	9	3.2	0.5
12T	10	3.2	0.6	9	4.2	1.4	9	3.3	0.5	8	3.1	0.5
13	17	3.0	1.3	5	2.2	0.3	11	2.9	0.9	7	3.1	0.8
14	8	4.0	1.6	13	3.2	0.8	10	2.8	1.2	10	3.0	0.4
15	8	1.9	0.7	10	2.7	1.4	10	2.9	0.6	9	3.1	1.3
16	9	3.5	1.5	10	3.7	0.8	10	2.9	0.8	10	2.6	0.9
16T	8	3.3	0.5	7	2.6	0.3	10	3.2	0.8	10	2.6	0.4
17	11	3.4	1.3	11	3.2	1.2	11	2.8	0.8	9	3.1	0.6
18	10	3.2	0.8	11	3.5	0.7	10	3.6	0.7	8	3.4	0.7
21	8	2.5	1.0	11	2.3	0.7	12	3.1	0.9	9	2.6	0.7
21T	10	2.5	0.5	7	2.8	0.5	10	3.0	1.0	11	2.8	0.9
22	11	2.4	0.6	10	3.6	0.9	8	3.1	0.9	10	3.4	0.5
23	10	3.5	0.8	10	2.5	0.7	10	2.7	0.7	10	3.4	1.3
24	8	4.2	1.1	9	3.2	0.8	12	2.9	0.8	10	3.1	0.7
25	10	2.6	0.7	11	3.0	1.2	9	2.0	0.8	10	2.8	0.7
31	10	3.3	1.3	7	3.0	0.8	10	3.0	0.5	10	3.0	0.8
33	10	3.5	1.1	10	3.2	0.3	12	3.0	1.0	11	3.1	0.6
34	9	3.8	1.5	10	3.2	0.8	10	3.2	0.6	10	3.2	0.9
41	10	3.3	0.9	10	2.8	0.8	11	2.8	0.8	10	3.4	0.8
42	7	2.8	0.7	10	2.7	1.4	10	3.3	1.1	10	3.4	0.8
43	8	2.4	0.6	11	2.6	0.7	10	3.1	0.5	11	3.1	0.6
43A	9	2.8	1.0	11	3.0	0.9	11	2.8	0.4	9	3.6	0.7
51	9	3.3	0.8	10	3.2	0.5	9	3.0	0.6	10	3.4	1.0
Project 1A												
12	6	4.8	0.5	9	4.7	0.9	10	4.9	1.9	10	5.0	0.7
12T	9	5.7	0.9	8	5.2	0.7	10	5.0	0.9	10	5.3	0.8
16	5	4.6	1.1	8	5.4	0.9	11	4.7	0.7	10	5.7	1.5
16T	12	5.2	1.7	8	6.4	0.8	9	4.8	1.1	11	5.8	1.1
21	6	6.2	0.8	8	5.3	0.9	11	5.3	0.9	9	5.1	1.4
21T	4	5.4	0.8	9	5.0	1.5	10	5.7	1.2	10	5.3	0.9
Project 1B												
11	7	3.0	0.9	5	3.4	0.8	10	3.3	1.0	8	3.8	0.8
11T	4	3.8	0.9	6	3.2	0.7	8	3.4	0.7	8	3.0	0.6
12	5	2.8	0.9	6	2.4	0.9	12	2.2	0.8	10	2.5	0.8
12T	3	2.5	0.6	5	2.7	1.0	8	2.3	0.8	12	3.0	1.3
16	4	2.4	0.6	5	2.4	1.1	7	3.1	0.5	11	3.1	1.4
16T	5	4.1	1.5	6	2.7	0.2	7	3.5	0.9	11	3.0	1.3
18	4	3.0	0.4	5	2.9	0.8	7	2.7	0.8	9	2.8	0.5
18T	5	2.0	0.6	5	3.0	0.9	11	2.6	0.9	10	3.3	0.6
21	5	3.4	0.8	6	2.6	0.7	10	2.9	0.9	10	2.6	0.6
21T	5	2.8	0.7	7	3.4	1.3	11	3.1	0.7	9	3.0	0.4
33	—	—	—	5	2.9	0.8	7	3.1	0.3	11	3.4	1.2
33T	5	3.5	1.2	5	3.1	1.0	9	2.8	0.3	8	4.2	1.3

*Number of tests
†Standard deviation

too wet or too dry and a test was needed to estimate the required adjustment in mixing water. The latter tests would normally be omitted in calculating averages and standard deviations, but the data do not distinguish between these tests and those made as routine. As a result averages are lowered at times, and raised at times, by inclusion of slumps obtained under these conditions. The standard deviations are increased by inclusion of these slumps.

Unit weight tests of fresh concrete were made during the placing of each round of each cement; usually there were two tests for each test section of

pavement. Data from these tests were used for calculating cement content and air content of concrete. In the case of cement content the accuracy of the unit weight measurement is adequate. However, the unit weight method of determining the amount of entrained air is not comparable in accuracy with methods developed since the construction of the test road, and it appears likely that the figures shown in Table 7-8 under "Approximate air" may be subject to errors of as much as ± 0.5.

Mixing

Two 27E single drum pavers were provided. It was the original intention to use both simultaneously, one for placing concrete below the reinforcing steel, the other for concrete above the reinforcing. This required that one paver operate on the finished subgrade between the forms and that the other operate outside the forms. Difficulties of maneuvering the batch trucks around the paver outside the forms led to the discontinuance of two-paver operation except for limited portions of the construction. In all cases the mixing time was 1¼ minutes with all materials in the mixer.

Placing and finishing

For the half width construction used, concrete below the reinforcing steel was deposited in three piles across the 12-ft width between forms. The concrete was spread by a steel strike-off blade sliding on the forms and pulled by the paver. A crew of shovelers ahead of the strike-off helped to spread the concrete. With the reinforcing steel in place the second strike-off was made by the transverse finishing machine carrying a reasonably heavy load ahead of the first screed. The transverse finishing machine had two reciprocating screeds but no vibrating or tamping equipment. In general, the finishing machine was not powerful enough to compact concrete of 2 to 3-in. slump with a single pass. In some cases several passes were required to close the surface and leave it in proper condition for the power operated longitudinal float which followed.

The longitudinal float had to do more work on the concrete than is usually expected of this equipment. There were frequent delays at times when the longitudinal float was unable to keep pace with the transverse finisher, and much concrete was carried by wheelbarrow from the point of deposit back to the area in which the longitudinal float was operating, to fill depressions left by the transverse finisher.

Relatively little work was done to the pavement surface after longitudinal floating. There was a limited amount of hand floating near the edges but no long handled float or straightedges were used. A plywood belt used without sawing motion was followed by a burlap drag while the concrete surface was still moist. Final surface texture was provided by drawing a stiff-fiber broom transversely across the pavement as the surface stiffened sufficiently to become crumbly under the broom.

Curing

Cotton-filled burlap mats applied to the concrete surface as soon as it had

stiffened sufficiently to support them without damage were used for curing. The mats were kept wet by sprinkling for three days. Frequent rains provided additional moisture after the required three days of moist curing had passed. The relative humidity ranged between 60 and 90 percent during much of the construction period, falling below 45 percent on only a few occasions. Temperatures were moderate, ranging from 51 to 80 F during work hours with a few lower readings at night.

WEATHER AND TRAFFIC CONDITIONS

As already mentioned, the test road is usually subjected to severe winter conditions including considerable snowfall with wide and frequent temperature fluctuations. It is in the center of a region which long has had the reputation of being hard on concrete. Table 7-10 gives the approximate number of cycles of freezing and thawing in the vicinity of the road during the past seven winters. With a few exceptions these figures were obtained from official Weather Bureau records at Alfred, N. Y., about 15 miles northeast of Wellsville. No official records are available for the location of the road itself. The figures show the number of days in each month during which the temperature was above 34 F and fell below 30 F at least once during the 24-hour period. The minimum total for the seven years is 570 cycles. The actual number of alternations of freezing and thawing during this period was probably considerably greater. These figures are, of course, only approximate. They do, however, give some idea of the amount of natural weathering on concrete structures in this region and to that extent may be considered pertinent.

State Route 17, on which the test sections are located, is a principal east and west through highway across the state. The average volume of traffic during 1949 for a typical 24-hour period was approximately 1900 vehicles. An idea of the type of traffic may be obtained from the following 8-hour count made on Thursday, September 22, 1949, at a loadometer station* near the east end of the project.

Type of vehicle	*Number*	
Passenger cars	1064	1064
Buses	13	
Trucks, single unit		
2 axle—4 tires	145	
2 axle—6 tires	138	
3 axle	16	
Trucks, combination		
3 axle	59	
4 axle	11	382
Total (8 hours)	1446	1446

Buses and trucks were approximately 25 percent of the total traffic using the road during the indicated period and approximately 20 percent of the trucks consisted of either 3- or 4-axle combinations. It will be seen, therefore, that this road carries a substantial volume of traffic, a fairly large percentage

*Data from New York State Highway Planning Survey Records.

TABLE 7-10—APPROXIMATE MINIMUM NUMBER OF CYCLES OF FREEZING AND THAWING ON NEW YORK TEST ROAD SINCE ITS CONSTRUCTION IN 1942*

Year	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Yearly Total
1942-43	2	3	15	14‡	8	11	13	15	3	84
1943-44	6	11	15	11	15	12	19	16	1	106
1944-45	1	13	8†	6	1	14	17	7	6	73
1945-46		5	14	9	8	11	12	16		75
1946-47	0	4	10	12	13	4	15	15	2	75
1947-48	4	4	16‡	11	4	10	12	9	5	75
1948-49	0		9	14	15	17	12	12	3	82
									Total	570

*Figures represent the number of days during month on which the temperature was above 34 F and fell below 30 F at least once during the 24-hour period. Data from Weather Bureau records at Alfred, N. Y.
†Data from Franklinville, N. Y.
‡Data from Elmira, N. Y.

of which is commercial. Some idea of the relative volume of commercial traffic using the road as compared to other main highways in New York State may be obtained from the fact that the average number of buses and trucks per 8-hour day (based on observations of 20 loadometer stations on the main highways of the state) was 524, as compared to 382 for the station on the test road.

An indication of the weight of some of the individual units may be obtained from loadometer records made during this same 8-hour period. Of the 369 trucks passing the station, 31 were selected by a process of sampling and weighed. The maximum total vehicle weight record was 60,220 lb, distributed on four axles, the maximum axle loading being 22,560 lb. Of the 31 units weighed, the maximum axle loading was over 20,000 lb in 11 cases. These heavy loads, together with rather unfavorable subgrade conditions on most of the road, no doubt account for the considerable amount of structural cracking that has occurred. This will be discussed more fully in a following section.

PERFORMANCE OF THE TEST SECTIONS

The experimental features of the test road are concerned entirely with durability, primarily the influence of cement quality upon durability. Durability in this case means the ability of the concrete as a *material* to resist the destructive effects of natural weathering as well as the effects of using chemicals for ice control in winter. It does not include the ability of the pavement as a structure to resist cracking due to traffic loads. Therefore, aside from the possible effect of cement quality upon initial shrinkage of concrete and therefore upon early cracking tendency, the structural behavior of the pavement is significant only to the extent to which excessive cracking might adversely affect ultimate durability. It will be of interest, therefore, in discussing the performance of various test sections to consider first, the extent of transverse cracking with special reference to the possible effect of cement composition or fineness upon early cracking tendency; second, the amount and severity of the "D-cracking" and the extent to which this type of cracking might have been influenced by the amount of structural cracking, if at all; and third, the type and amount of surface scale, particularly on Project 1B.

TRANSVERSE CRACKING

A preliminary inspection of the road was made in October-November 1942, when the test sections were less than three months old. This was to note the amount of early cracking as well as to record general surface conditions, local areas of defective concrete, etc. Unfortunately, a number of sections in Project 1, principally in Rounds 2 and 3, were not included in this first survey. However, none of the sections inspected in November 1942, including all sections in Projects 1A and 1B, showed any transverse cracks. Also, of the 33 sections in Project 1 for which there is no record as of November 1942, 25 showed no cracks at the next inspection made in May 1943. This leaves only eight sections of 168 which might possibly have cracked as early as November 1942. It is reasonable to assume, therefore, that early cracking must have been almost completely nonexistent.

This lack of early cracking is interesting, especially in view of the wide variations in composition and fineness of the cements used. For example, as shown in Table 7-2, the computed C_3S content varied from a high of 65.5 percent (Cement 15) to a low of 20.0 percent (Cement 41); the computed C_3A content from a high of 13.2 percent (Cement 18T) to a low of 3.4 percent (Cement 42) and the fineness, as indicated by the Wagner turbidimeter, from a high of 2795 sq cm per g (Cement 31) to a low of 1630 sq cm per g (Cement 21). The fact that practically no transverse cracking developed during this early period would indicate that differences in the volume change characteristics of the various cements, although possibly of considerable magnitude from a theoretical point of view, were not of practical importance as influencing the early cracking tendency of the concrete.

The total number of full and partial transverse cracks appearing in the test sections when inspected in May 1943, May 1945, May 1947 and August 1949 are shown in Table 7-11 for Project 1 and in Table 7-12 for Projects 1A and 1B. The figures in each case indicate the total number of cracks appearing in the two 75-ft slabs comprising each test section and do not include the transverse joint between the slabs. Note that some transverse cracking had occurred by May 1943, that is after the first winter, and also that the average amount of cracking has increased progressively since that time. The average uncracked slab length, including joints, over the entire length of each round of each project is shown as the last figure in each vertical column of the tables. Averaging the values for the four rounds in each project it will be seen that, in Project 1 (Table 7-11), the average slab length has decreased from 62 ft in 1943 to 28 ft in 1949. The corresponding figures for Project 1A (Table 7-12) are 67 and 28 ft whereas for Project 1B they are 73 and 63 ft.

The freedom from transverse cracking on Project 1B is undoubtedly because this entire portion of the project is on a continuous grade averaging about 4 percent (Fig. 7-5) whereas Projects 1 and 1A lie relatively flat with comparatively poor natural drainage (Fig. 7-4). That this is the predominant reason for the difference in cracking is further borne out by the fact that Rounds 2 and 4 of Project 1, portions of which were built on a slight grade

TABLE 7-11—NUMBER OF FULL AND PARTIAL TRANSVERSE CRACKS AS OF MONTH AND YEAR INDICATED*

Project 1

Cement No.	Round 1				Round 2				Round 3				Round 4			
	May 1943	May 1945	May 1947	Aug. 1949	May 1943	May 1945	May 1947	Aug. 1949	May 1943	May 1945	May 1947	Aug. 1949	May 1943	May 1945	May 1947	Aug. 1949
11	0	1	1	4	1	1	2	3	2	3	5	6	0	0	0	0
12	2	2	3	7	2	3	4	4	2	3	4	4	0	0	0	1
12T	0	0	0	1	1	2	2	2	0	0	0	1	3	5	5†	5
13	0	0	0	4	0	0	1	2	3	4	5	6	0	0	4	8
14	0	0	0	2	0	0	0	1	1	1	7	8	0	0	1	4
15	1	2	2†	2†	0	0	4	4	0	0	3	6	0	0	0	1
16	2	2	3	3	0	0	2	2	0	0	1	3	0	1	2	4
16T	0	0	1	4	0	0	0	2	1	1	2	2	0	2	3	3
17	1	2	2	5	0	4	9	9	0	0	0	0	0	0	1	1
18	0	0	0	8	0	0	4	4	0	0	0	0	0	1	2	2
21	1	1	1	7	0	0	1	1	0	1	3	4	0	0	1	1
21T	0	0	0	0	1	2	2	2	0	0	0	0	0	1	1	1
22	0	0	0	2	0	0	0	0	0	2	6†	7†	0	0	2	2
23	1	6	6†	7†	0	2	5	5	2	3	2	2	0	0	1	2
24	0	0	0	6	0	0	0	0	1	1	2	2	0	0	0	0
25	0	0	1	2	0	0	1	3	2	2	2	2	0	1	2	3
31	2	6	6	9	0	0	2	2	1	3	4	5	0	0	5	5
33	1	3	3	7	0	0	4	4	2	2	3	3	0	1	5	5
34	2	4	7	12	0	2	2	2	2	2	5	5	0	0	1	1
41	0	2	3	4	0	2	3	3	0	2	7	9	0	0	1	4
42	0	2	4	6	0	0	1	2	0	2	5	6	–‡	1	2	2
43	1	1	2	3	0	0	0	3	0	1	6	8	0	3	4	6
43A	2	4	8	13	0	4	6	7	0	0	0	0	0	0	2	3
51	0	1	2	6	0	0	0	0	0	1	1	2	0	0	1	2
Total	16	39	55	124	5	22	55	67	19	34	73	91	3	16	46	66
Ave. §	56	41	35	21	68	51	35	31	54	44	30	26	70	56	38	32

*Figures show total number of full and partial transverse cracks which have appeared in the two 75-ft sections containing each cement in each round.
Only partial transverse cracks more than 2 ft in length are included.
Corner breaks and short branch cracks not included.
†Indicates that one of the 75-ft slabs in this section is on approach slab to a bridge and that a portion of the approach slab has been covered with bituminous material. Only those cracks which appear in the uncovered portion are indicated in the tabulation.
‡No record.
§Average spacing, in ft, of cracks and joints for all cements in each round. Total length of each round: 3600 ft.

at the west end of the project show somewhat longer average slab lengths than either Rounds 1 or 3, both of which were constructed on the flat section lying in the central portion of the project.

There appears to be no correlation whatever between transverse cracking and either individual cements or cement types. The variations from round to round for a given cement are as great as the variations between cements. For example, referring again to Table 7-11, the section containing Cement 17, Round 2, showed nine transverse cracks in August 1949 whereas this same cement in Round 3 was entirely free from cracking. Again, the section containing Cement 21, Round 1, had seven cracks whereas Rounds 2 and 4 with this same cement showed only one crack. The extreme variation is probably in the sections containing Cement 43A. In this case, Round 1 showed 13 cracks whereas Round 3 was entirely free from cracking. These wide individual variations which, as has been stated, are no doubt due primarily to variations in subgrade support, would make any conclusion regarding the effect of either cement type or individual cement upon cracking extremely questionable.

TABLE 7-12—NUMBER OF FULL AND PARTIAL TRANSVERSE CRACKS AS OF MONTH AND YEAR INDICATED*

Project 1A

Cement No.	Round 1				Round 2				Round 3				Round 4			
	May 1943	May 1945	May 1947	Aug. 1949	May 1943	May 1945	May 1947	Aug. 1949	May 1943	May 1945	May 1947	Aug. 1949	May 1943	May 1945	May 1947	Aug. 1949
12	0	1	1	4	0	1	1	5	–†	0	0	0	0	1	1	3
12T	0	0	0	3	0	1	1	3	0	0	3	6	2	3	3	3
16	0	0	0	9	0	0	3	6	0	0	4	5	0	0	0	1
16T	1	2	2	9	0	0	1	2	0	0	1	2	3	3	5	6
21	0	0	1	3	0	0	0	0	0	0	1	1	0	0	0	0
21T	0	0	2	9	0	0	0	0	0	0	4	7	1	1	1	2
Total	1	3	6	37	0	2	6	16	0	0	13	21	6	8	10	15
Ave.‡	69	60	50	18	75	64	50	32	75	75	36	27	50	45	41	33

Project 1B

11	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0
11T	0	0	0	0	0	0	1	2	1	2	2	2	0	0	1	1
12	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
12T	0	0	0	0	0	0	0	0	–†	0	0	0	0	0	0	1
16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
16T	0	0	0	0	0	1	1	1	–†	0	0	0	0	0	0	0
18	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0
18T	0	1	2	2	0	0	0	0	0	0	0	0	0	0	0	0
21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
21T	0	0	0	0	0	0	0	0	–†	0	0	0	0	0	0	0
33	0	0	0	1	0	0	0	0	0	0	1	1	0	0	0	1
33T	0	0	0	0	0	0	0	2	1	1	2	2	0	0	0	0
Total	0	1	3	5	0	1	2	5	2	3	5	6	0	0	1	3
Ave.‡	75	72	67	62	75	72	69	62	68	67	62	60	75	75	72	67

*Figures show the total number of full and partial transverse cracks which have appeared in the two 75-ft sections containing each cement in each round.
Only partial transverse cracks more than 2 ft in length are included.
Corner breaks and short branch cracks not included.
†No record.
‡Average spacing of cracks and joints for all cements in each round. Total length of round: Project 1A, 900 ft, Project 1B, 1800 ft.

The freedom from cracking on Project 1B as compared to Projects 1 and 1A illustrates strikingly the necessity for adequate subgrade drainage for control of structural cracking.

Scattered plastic shrinkage cracks were noted on many of the sections. These are fine surface cracks up to 6 in. or more in length, roughly parallel and usually lying at an angle of about 45° with the longitudinal axis of the road. The occurrence of these cracks could not be related to any variable, either cement, weather conditions or other factors. They have had no adverse effect on the durability of the concrete.

D-CRACKING

In a concrete pavement, deterioration due to weathering is usually evidenced first by the appearance of fine, more or less closely spaced parallel surface cracks along the joints and structural cracks and sometimes along the free edges of the pavement slab. These cracks are frequently filled with a dark colored deposit—principally calcium carbonate—whence the name "D-cracks" or "Deposit cracks." Concrete in which D-cracks have formed usually is low in strength. The matrix is dull and chalky, in contrast to the dense, blue-gray matrix usually associated with sound concrete. In pave-

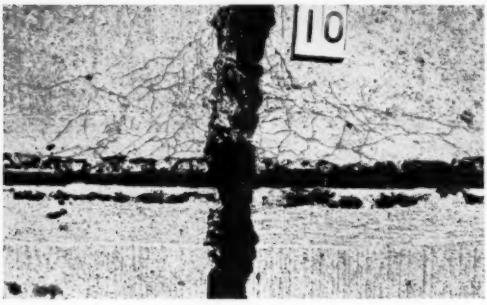

Fig. 7-7—Cracking at intersection of transverse and longitudinal joints

ments, D-cracks along joints are frequently but not always followed by progressive spalling and raveling at the joint, necessitating continuous and expensive maintenance. Deterioration at the intersection of a transverse joint with the longitudinal joint, probably the result of initial D-cracking, is shown in Fig. 7-7. Note the entire absence of cracks in the concrete on the opposite side of the longitudinal joint.

In view of the significance of D-cracking as an indication of lack of durability, considerable attention has been devoted to recording the development of this type of cracking on the test road. Complete notes were taken during the various official inspections and these notes on D-cracking are condensed in Table 7-13 showing for each of four inspections, the sections in Project 1 which showed any evidence of D-cracking on the dates indicated and corresponding data for Projects 1A and 1B.

In studying these data it will be immediately evident that, in some instances, well-defined D-cracks are recorded as having been noted on certain inspections with no reference being made on following inspections to their occurrence. Examples in Project 1 (Table 7-13) are Round 3 of Cement 11 and Round 3 of Cement 43A. In these and similar cases it is probable that the original D-crack pattern has been obscured by later spalling or raveling (Fig. 7-8). However, the absence of any notes on D-cracking at the time of later inspections would indicate that the original D-cracking was not progressive. In other cases where a trace of D-cracking was indicated during an early inspection with no reference made at the time of later inspections (for example, Cements 12 and 17 in Round 1 and Cement 23 in Round 3), it is possible that the observers were mistaken and were recording as D-cracks faint stress cracks caused by external loading. This is probably true of fine cracks along the free edges of the slabs which were first thought to be D-cracks. Cracks of this nature have now practically disappeared as the edges of the slabs have become rounded off by traffic.

It is not surprising that there should be some individual inconsistencies in a record of this kind. The inspections were made in all kinds of weather. Incipient D-cracking, plainly visible under certain conditions, might not be visible at all under other conditions. Therefore, faint indications of D-cracking

TABLE 7-13—SECTIONS SHOWING EVIDENCE OF D-CRACKING ON DATES INDICATED

Project 1

Cement No.	Round 1				Round 2				Round 3				Round 4			
	May 1943	May 1945	May 1947	June 1949	May 1943	May 1945	May 1947	June 1949	May 1943	May 1945	May 1947	June 1949	May 1943	May 1945	May 1947	June 1949
11	O	O	O	O	O	O	T	O	X	O	O	O	O	O	O	O
12	O	T	O	O	O	O	O	O	O	O	X	X	O	O	O	O
12T	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O
13	O	O	O	O	O	O	O	O	O	O	O	T	O	O	O	O
14	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O
15	O	X	O	O	O	O	O	O	O	O	O	X	O	O	O	T
16	X	O	X	O	O	O	X	X	O	O	O	X	O	O	O	O
16T	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	X
17	O	T	O	O	O	O	O	O	O	O	O	O	O	O	O	O
18	O	O	O	O	O	O	O	O	O	O	O	T	O	O	O	O
21	X	X	X	X	X	X	X	X	O	O	O	O	O	O	O	T
21T	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O
22	O	O	O	O	O	O	X	X	O	O	O	O	O	O	O	O
23	O	X	X	X	O	O	O	O	T	O	O	O	O	O	O	O
24	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	T
25	O	O	O	O	O	O	T	O	O	O	O	O	O	O	X	X
31	O	X	X	X	X	T	X	X	O	O	O	O	O	O	O	O
33	O	O	T	O	O	O	O	O	O	O	O	O	O	O	O	O
34	O	O	T	O	O	O	O	T	O	O	O	O	O	O	O	O
41	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O
42	O	O	O	O	X	T	X	X	O	O	O	O	–	O	O	O
43	O	O	O	O	O	O	O	O	O	O	T	O	O	O	X	X
43A	X	X	X	O	O	O	X	X	X	O	O	O	O	O	O	X
51	O	T	T	O	O	O	O	O	O	O	O	O	O	O	O	O

Project 1A

12	O	T	O	O	O	O	T	O	–	O	O	O	O	O	O	O
12T	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O
16	O	T	T	O	O	O	O	O	O	O	O	O	O	O	O	O
16T	O	T	O	O	O	O	O	O	O	O	O	O	O	O	O	O
21	O	X	X	X	O	X	X	X	X	X	X	X	X	X	X	X
21T	O	O	O	O	O	O	X	X	O	O	O	O	O	O	O	O

Project 1B

11	O	O	O	O	O	O	O	O	O	O	X	X	O	O	O	O
11T	O	O	O	O	O	O	X	O	O	O	O	O	O	O	O	O
12	O	O	O	O	T	T	O	T	O	O	O	O	O	X	O	O
12T	O	O	T	O	O	O	O	O	–	O	O	O	O	O	O	O
16	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O
16T	O	O	O	O	O	O	O	O	–	O	O	O	O	O	O	O
18	O	O	O	O	T	O	O	O	O	O	O	O	O	O	O	O
18T	O	O	T	O	O	O	O	O	O	O	O	O	O	O	X	O
21	X	X	X	X	X	X	X	X	X	X	X	X	O	X	X	X
21T	T	O	O	O	O	T	O	O	–	O	O	O	O	O	O	O
33	O	O	O	O	O	O	O	O	O	O	O	O	O	O	X	X
33T	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O

O—no evidence of D-cracking on date indicated
T—trace (sometimes reported as faint) D-crack pattern along joints or cracks or both.
X—well defined D-crack pattern along joints or cracks or both.
– –no record.
Each section in each round 150 ft long (two 75-ft panels).

should not be considered significant in cases where the cracks could not be located during future inspections. On the other hand, in cases where the notes indicate that well-defined D-cracking was observed at a joint or crack on two or more consecutive inspections, it may generally be assumed that the cracking is significant and that the trouble is probably progressive. Fig. 7-9 is a view of a transverse joint which shows no evidence of D-cracking or spalling notwithstanding relatively poor maintenance.

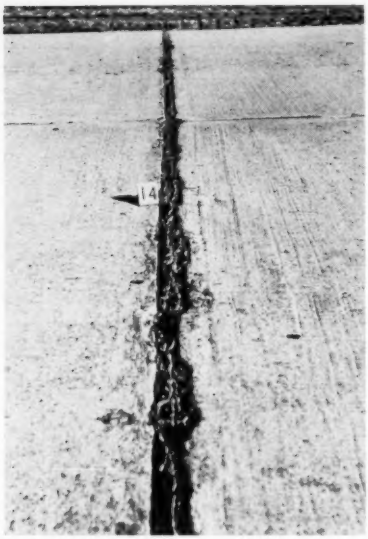

Fig. 7-8—Spalling along transverse joint

Fig. 7-9—View along transverse joint. Concrete in excellent condition despite lack of maintenance

Classified on this basis, significant D-cracking has occurred on Project 1 in two out of four rounds with three cements; in one out of four rounds with seven cements and has not occurred at all with fourteen cements. In Project 1A, significant D-cracking has developed in all four rounds with one cement; in one out of four rounds with one cement, with no significant D-cracking otherwise. In Project 1B, one cement shows significant D-cracking in all four rounds; two cements in one round with no significant D-cracking in the case of the other nine cements.

Reference to Table 7-13 will show that Cement 21, used in all three projects, has shown a greater tendency to develop D-cracking than any other. Significant D-cracking has appeared on 10 of the 12 sections containing this cement. However, even in this case there are two rounds of sections (Rounds 3 and 4 of Project 1) which are so far free from this defect. It is also true that on other projects of the Long-Time Study, this cement ranks among the best of those under study. No satisfactory explanation has, as yet, been advanced to account for the spotty performance of this particular cement.

The only other cements which show significant D-cracking in more than one round are Cements 31 and 43A, both in Project 1. Cement 31 shows well defined D-cracking in Rounds 1 and 2 with no evidence of this defect in Rounds 3 and 4. Cement 43A shows significant D-cracking in Rounds 1 and 2 with some indications in Rounds 3 and 4.

In general, apart from Cement 21, there is no relation between individual cements or cement types and D-cracking. Cement 21 does show this tendency definitely on the test road. On the other hand, the air-entraining counter-

part of this cement, 21T, is almost completely free from this defect. In fact, little D-cracking has developed on any section containing any air-entraining cement. Only nine of the 48 sections containing these cements show any evidence of D-cracking whereas 51 out of the 120 sections containing the plain cements are affected to some degree.

A detailed comparison of data on structural cracking (Tables 7-11 and 7-12) with data on D-cracking just discussed fails to reveal any relation whatever. It was thought that a section which had cracked excessively due to structural failure might tend to develop more D-cracks than one which had not cracked or on which only a moderate amount of cracking had occurred. This would be due to the fact that D-cracks tend to form along structural cracks as well as along joints. However, the data do not bear this out. For example, in Project 1, Cement 41 is entirely free from D-cracks whereas the structural cracking with this cement was as high as with the others. On the other hand, in Projects 1A and 1B, Cement 21 was quite free from structural cracking, whereas this cement shows a marked tendency to develop D-cracks.

SURFACE SCALING

The outstanding development to date is the effect of air entrainment on resistance to scaling. This is strikingly shown in Project 1B, and convincingly though less spectacularly verified in portions of Project 1. In Project 1B six air-entraining cements and their six non-air-entraining counterparts were used in three-quarters of a mile of pavement where the application of abrasives treated with chemicals for ice control was anticipated. This length of pavement has grades up to 4.8 percent and two long-radius curves with slight super-elevation. It has received ice control treatment as indicated by Table 7-14 and Fig. 7-4. The quantities of materials shown in the table are averages for the total length of pavement treated and do not necessarily represent the applications made at any one location.

Of the 24 test sections with non-air-entraining cements there is scaling in some degree on all but one. On the 23 affected sections the amount of scaling varies from traces only to nearly 40 percent of the total area in some cases.

TABLE 7-14—SUMMARY OF ICE CONTROL TREATMENT 1942-43 TO 1948-49
Project 1B

	1942-43	1943-44	1944-45	1945-46	1946-47	1947-48	1948-49
Total No. applications of abrasive	72	65	71	48	61	67	39
No. applications abrasive with $CaCl_2$		0	53	40	0	0	0
No. applications abrasive with $NaCl$	0	0	18	8	53	22	0
Abrasives per 100 ft of pavement, tons (Ave.)	4.7	4.7	4.0	1.1	1.1	1.0	1.4
$CaCl_2$ per 100 ft, lb (Ave.)	18.5	0	135.8	35.2	0	0	0
$NaCl$ per 100 ft, lb (Ave.)	0	0	24.5	7.8	15.8	5.7	0

TABLE 7-15—PAVEMENT SCALING ON PROJECT 1B—PERCENT TOTAL AREA SCALED AS OF AUGUST, 1949

Cement No.	Non-air-entraining cement					Air-entraining cement				
	1	2	3	4	Ave.	1	2	3	4	Ave.
11	25	1	1	T	7	0	0	0	0	0
12	38	5	T	9	13	0	0	0	0	0
16	35	2	2	2	10	0	0	0	0	0
18	20	1	1	4	6	0	0	0	0	0
21	10	1	T	3	4	0	0	0	0	0
33	22	4	0	T	6	0	0	0	0	0
Ave.					8					0

T—trace

For the air-entraining cements there is no scaling on any of the 24 test sections. A detailed comparison of the performance of non-air-entrained and air-entrained pavement is shown in Table 7-15. The most severe scaling was on slabs of Round 1. It is presumed that this is due to heavy applications of chemically treated abrasives on the east bound (south) lane at the steepest portion of the grade (Fig. 7-5). Fig. 7-10 is a general view of Project 1B showing the type of surface scale which has so far developed.

On Project 1, where the use of chemicals for ice control had not been anticipated, scaling is confined to relatively short areas near intersections on which applications of chemically treated abrasives have been required for traffic safety. One such area is located at Station 335 (Fig. 7-4). An intersection with stoplights and a public school combine to demand liberal use of ice control materials. On the westbound (north) lane the two air-entraining sections 12T-5 and 16T-4 receive the heaviest applications but show no scaling. Sections 21T-4 and 25-4 within possible carry-over range also show no scaling. On the eastbound (south) lane test sections 18-2, 15-2, 11-2 and 13-2 are within the likely area of ice control applications with 13-2 nearest the intersection and probably subjected to the heaviest applications. On these sections (all non-air-entraining) the percent scaling is: 18-2, 0; 15-2, 2; 11-2, 10, and 13-2, 25. Because the quantity of ice control materials used would normally increase toward the intersection it is believed that the amount of scaling reflects, roughly, the relative amounts of chemicals applied to these non-air-entraining slabs. Since the north lane, of air-entraining concrete, has no scale and the south lane, of non-air-entraining concrete, is scaled appreci-

Fig. 7-10—General view, Project 1B

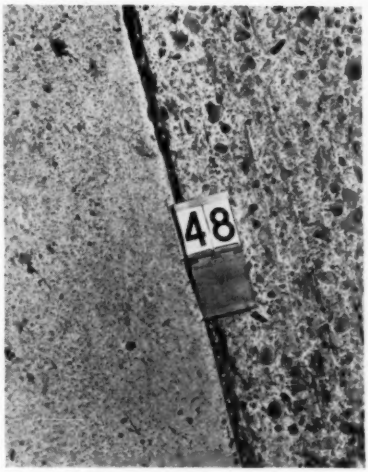

Fig. 7-11—Project 1 near Sta. 330. Air-entraining cement used on left; non-air-entraining cement on right

ably in comparable areas, the effect of air-entrainment, clearly demonstrated in Project 1B, is further verified (Fig. 7-11).

EFFECT OF USING ADDED MIXING WATER

The only direct comparison of the effect of mix proportions is provided by Projects 1 and 1A. The principal intent of Project 1A was to show the effect of added mixing water. For this purpose six of the 24 cements used in Project 1 three air entraining and three non-air entraining) were also used in Project 1A with the slump increased by slightly less than 3 in. This corresponded to an increase in water-cement ratio of approximately ⅓ gal. per sack of cement. Thus far no perceptible differences in performance of comparable sections of Projects 1 and 1A have developed.

SUMMARY AND CONCLUSIONS

No conclusive indications can be drawn from observations on the New York Test Road that any one of the 21 plain or non-air-entraining test cements is superior to another. However, all six of the air-entraining cements are definitely superior to the non-air-entraining cements in resistance to surface scaling. Air entrainment appears also to have improved resistance to D-cracking, especially in the case of Cement 21. It is the conviction of the writers that as of this time the performance of the test pavement reveals no clear-cut advantage for any of the test cements or for any of the five types other than would be obtained by the use of air entrainment.

Entirely apart from the question of cement performance, the freedom from transverse cracking on Project 1B illustrates strikingly the advantage of good subgrade drainage in controlling structural cracking.

The conclusions which may be drawn from this experimental project after seven and one-half years service are:

1. The effects of air entrainment on improving resistance of the pavement to scaling and weathering overshadow all other variables.

2. The use of abrasives with ice control chemicals caused scaling on the non-air-entraining pavement sections.

3. An increase in water content of concrete of approximately ⅓ gal. per bag (2 to 3 in. difference in slump) has had no noticeable effect on the performance of the pavement.

4. Structural cracking of pavement slabs, now assuming serious proportions, is associated with the supporting power of the subgrade and not with cement performance.

5. Except as noted under 1 and 2, no significant differences in performance have developed between individual cements or between types of cement.

ACKNOWLEDGMENTS

The writers thank H. F. Brumm, district engineer, State of New York Department of Public Works, and his associates for their cooperation in the construction of the New York Test Road and their assistance in making the inspections and assembling record data.

Discussion of a paper by F. H. Jackson and I. L. Tyler:

Long-Time Study of Cement Performance in Concrete—Chapter 7. New York Test Road*

By M. SPINDEL†

Although the performance of the test sections was assessed only with respect to "durability of the concrete as a material," this comprises many properties required of concrete in general and of pavement concrete in particular. The various properties have been investigated and tested for tens of years in laboratories and in the field, culminating in the Long-Time Study of Cement Performance in Concrete carried out in the United States.

The writer would like to discuss from the historical point of view one aspect stressed by the authors. This is the role of special cements, *i.e.*, portland cements differing in chemical composition to obtain different improved properties of concrete, compared with the role of admixtures added in very small quantities to concrete made of conventional or special cements.

In 1944 the writer referred‡ to his efforts to avoid further damage to the reputation and use of portland cement and concrete due to failures of concrete structures, especially large dams and other hydraulic structures, pipes and pavements. These failures were discussed at the Congress of the IATM in Switzerland in 1931 and at the International Congress on Large Dams in Sweden in 1933. Leading experts on cement and concrete, as well as those on large dams, then came to the conclusion that the damage was due only to the use of modern portland cements which, in their opinion, were not suitable for concrete exposed to severe water and weather conditions.

The cement specialists who had just succeeded in explaining the role of the different cement compounds—the so-called clinker minerals—during manufacture and hydration were confident that this improved knowledge could be used successfully for the manufacture of new types of special cements which would satisfy the various special requirements for concrete structures. The cement manufacturers, especially in the United States and Sweden, were apparently anxious to avoid the damage which occurred with conventional portland cements and thus the epoch of special portland cements started with an International Special Committee on this question which for years made valuable contributions to the further development and testing of cement and concrete.

*ACI JOURNAL, June 1951, *Proc.* V. 47, p. 773. Disc. 47-51 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 4, Dec. 1951, *Proceedings* V. 47.

†Research Engineer and Consultant, London, England.

‡Spindel, M., "Concrete and its Improvement, The Role of Admixtures," *The Builder*, Jan. 21, 1944.

Nevertheless, the significance and necessity of special cements and the disadvantages of the usual portland cements were greatly overstressed everywhere. Dr. Forsen, the cement scientist of the leading Swedish cement works, for example, published a paper* containing a table showing that ordinary portland cements and high-early strength cements were suitable only for structures such as houses, while all other types, especially hydraulic structures in which he expressly included concrete pavements, had to be made of special cements only. This table was illustrated by a diagram showing in the known composition triangle the chemical position of the Swedish cements, special portland cement for hydraulic structures, ordinary portland cement and high-early strength portland cement. Apart from an experimental Swedish special cement *H* the most interesting feature of this diagram was the sharp chemical "boundary line" between those portland cements indicated as suitable and not suitable for concrete pavements. This boundary line was based by Dr. Forsen on investigations in the United States by Ira Paul.

The writer strongly opposed these views in a lecture in 1935 which was published in several periodicals.† On these and other occasions the writer pointed out that damage to concrete was not due to modern portland cements but because the concrete in question was made on wrong principles. As to new types of special cements certain properties of concrete could be improved by their use but only at the expense of others, such as strength. Therefore he concluded that it was not possible to manufacture a new type of portland cement which would combine all advantages, but better results could have been and still could be obtained by suitable additions to cement and concrete.

In the United States three types of special portland cements, *i.e.*, Types II, IV and V, were designed, standardized, manufactured, tested and used advantageously, as far as possible, by experts of great experience. At the same time, or rather since about 1940, the previous prejudice of cement manufacturers against admixtures to be added to cement and concrete changed to confidence in their advantageous effect. Initially it was the effect of air-entraining agents and air entrainment on resistance to frost and aggressive substances which impressed all concerned.‡

Having had the privilege of participating from time to time in discussions on this subject, the writer was most interested in the authors' statement that "no one cement proved superior to the others tested. However, the effects of air entrainment in improving resistance of the pavement to scaling and weathering overshadowed all other variables." Apart from the original Types I and III portland cements, Types II, IV and V were used in this long-time study and in spite of all the differences in chemical composition intended to improve properties of the concrete these were negligible com-

Zement, No. 13, 1935.
†*Beton u Eisen*, No. 17, 1935; *Zement*, No. 14, 1936; "Special Cements," International Congress on Large Dams, Washington, D. C., 1936.
‡Bates, P. H., "Portland Cement—Theories (Proven and Otherwise) and Specifications," *Proceedings* ASTM, V. 40, 1940, p. 469.

pared with the advantages obtained with a suitable admixture to ordinary portland cements.

One might object that only Type II and perhaps also Type IV cement were the special cements which matter for pavements but regarding these we have to consider also the remarks of the co-author, I. L. Tyler, in one of his previous discussions* when he stated, "In my own case nearly ten years of satisfactory experience with Type II and Type IV cements exclusively in a very considerable volume of concrete left a fondness for these types of cement which amounted to a distinct prejudice. On an unbiased study of the record, however, I have been unable to find any significant differences in the performances of Type I and Type II cements except in the case of mass concrete or concrete exposed to sulfate-bearing soils or waters." What a difference between this statement made in 1949 by a competent American cement expert and the boundary line in the diagram published in Europe in 1935.

The forementioned statements by the authors do not, of course, mean that the quality of the cement used for concrete pavements does not matter at all. On the contrary, it is of great importance. In discussion of a paper† it was pointed out that for concrete roads the best portland cement available having the necessary special qualities had to be used and not to be satisfied merely with the minimum requirements of the standard cement specifications. The same he remarked, was true and even more important with regard to quality of aggregates, design and control of mixes, and placing and curing of the concrete. It might now be added that the control of air-entraining concrete is usually and has to be much more severe than that of ordinary concrete, requiring the determination of the percentages of cement, aggregates, water and air in absolute volumes and, as far as possible, also the size, shape and distribution of air pores.

Although the advantages of air-entraining concrete appear definitely established, at least as far as resistance to frost and scaling of pavement concrete is concerned, there is still another question to be answered sooner or later. Why confine such studies and statements of favorable results to air-entraining concrete?

B. D. Tallamy‡ dealt with the use of dilute oil applications as a protective measure to eliminate capillary action in concrete. Even more favorable results were reported later.§ For example, Ira Paul described use of a heavy asphaltic oil residue as a waterproofing medium for portland cement concrete. His discussion stressed the necessity of keeping water out of concrete, or preventing it from getting in, which means dense concrete in the first place.

In this direction we must consider those admixtures which reduce water-

*Tyler, I. L., Discussion of "Experience with Air-Entraining Concrete in New Jersey," Part 2, Dec. 1949 ACI JOURNAL, p. 528-11.

†Spindel, M., Discussion of "The Concrete Road," by F. N. Sparkes and A. F. Smith, Road Engineering Division, Institution of Civil Engineers, London, 1945.

‡Tallamy, B. D., "Control of Concrete Pavement Scaling Caused by Chloride Salts," ACI JOURNAL, Mar. 1949, *Proc.* V. 45, p. 513.

§Discussion of "Control of Concrete Pavement Scaling Caused by Chloride Salts," Part 2, Dec. 1949 ACI JOURNAL, p. 520-1.

cement ratio and at the same time reduce rather than increase the void-cement ratio. According to a paper by Major Willard C. Jensen* an admixture for that purpose was used successfully by a U. S. Engineer Aviation Battalion for pavement concrete of a specified minimum compressive strength of 4975 psi and flexural strength of 683 psi.

From the above it may be seen that, although very good results were obtained with air-entraining concrete, this is only one of the means found suitable for making concrete resistant to frost and to scaling caused by de-icing chemicals. Some further studies to compare resistance and durability of air-entraining concrete with concrete made with other admixtures which have already stood the test of time would no doubt contribute to the appreciation of the important work done to improve concrete pavements.

*Jensen, Willard C., "Airlift Airfield Gets Heavy Concrete Runway," *Civil Engineering*, Oct. 1950, p. 21.

Title No. 47-52

Slab Warping Affects Pavement Joint Performance*

By F. N. HVEEM†

SYNOPSIS

An investigation of joint troubles and failure is discussed and a method of determining thermal and moisture expansion in thin concrete specimens described. Evidence of warping and curling of pavement slabs and the sequence of events leading to pumping and subsequent faulting of joints are considered. It was evident that curling was due to combined effects of temperature differential and moisture. Profilograph studies of pavement surfaces are described.

One solution to pavement failures is the elimination of expansion joints and the spacing of contraction joints as far apart as possible. No practical method is available that will prevent moisture from accumulating beneath the pavement.

INTRODUCTION

By 1944 highway engineers throughout the United States were becoming increasingly concerned over the evidences of distress in some portland cement concrete pavements, generally in the form of mud pumping or faulting at the joints. At this time the California Division of Highways began extensive field and laboratory studies to determine the causes for troubles at the joints in California concrete pavements. These studies, covering four years, included taking samples of subgrade soils through holes cut in the pavement by a core drill, inspection of the condition of dowels where faulting was in evidence, tabulation of construction data including design features and sources of materials and a scrutiny of any facts or data which might have a bearing upon the problem. As a result, certain remedial measures were put into effect to reduce the probability that similar types of distress would develop on future pavements.

Concrete pavements were constructed in California as early as 1914, and the use of concrete along with other materials has continued with periodical changes in design, including greater pavement width and thickness. Increasing attention has been given to the selection and compaction of materials in the supporting subgrade.

Prior to 1925 most concrete pavements were placed in continuous slabs without joints or with construction joints placed only at the end of each

*Presented at the ACI 47th annual convention, San Francisco, Feb. 20, 1951. Title No. 47-52 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 10, June 1951, *Proceedings* V. 47. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1951. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Materials and Research Engineer, State of California Division of Highways, Sacramento.

day's run. Frequent buckles or blowups in these early thin pavements directed attention to the expansive forces developed during hot weather.

To prevent accumulation of expansive movements in pavements, expansion joints were installed by most states, and to discipline the inevitable cracks into a uniform and more sightly pattern, contraction joints were introduced. In line with trends throughout the United States, the typical California pavement design from 1925 to 1940 included expansion joints at 60-ft intervals with two contraction or dummy joints placed between each expansion joint, thus providing a 20-ft slab.

The introduction of expansion and contraction joints led to concern over the weakness imparted to the pavement at these joints, and load transfer devices, generally in the form of dowels, were soon proposed. California adopted round dowels of hard-grade steel placed first at 28-in. intervals and later a greater number were used at 15-in. spacing. With the steady increase in traffic, engineers began to observe faulting or vertical offsets at expansion joints. Because of the attention focused on expansive properties of concrete, these vertical offsets or stepoffs were first attributed to careless construction, on the theory that the expansion joint materials had become canted or tipped from the vertical, thus providing an incline which resulted in an offset when the slabs were subjected to longitudinal thrusts. While this type of faulting does exist, investigation of California pavements indicated that it is relatively infrequent.

PROFILOGRAPH

With the increase of traffic and rapid multiplication of heavy trucks with three or more axles, faulting became more marked. To study the surface contour of pavement and to detect minute changes in riding qualities, a profilograph was constructed in the materials and research department.* The profilograph measures the intimate profile of a pavement and makes a direct record in the form of a graph on a dependably accurate scale. To check the accuracy of the instrument, profilograph records of pavement surfaces were compared to profiles obtained by scale measurement from a stretched piano wire. The relation between these two methods of measurement is shown in Fig. 1.

A few trial runs with the profilograph indicated that many concrete pavements were elevated or "turned up" at each joint or transverse crack; this was true even though the individual pavement had been known to be markedly smooth when constructed. To follow and chart the changes that characteristically occur in a concrete pavement after construction, a project under construction in 1944 was selected and profilograph records taken of the newly laid pavement as soon as the surface had hardened sufficiently. Fig. 2A shows the profilograph of a section placed June 8.

The newly finished pavement was very smooth; surface irregularities were so small as to be negligible (less than $\frac{1}{32}$ in. in 10 ft). This smooth condition

*California Highways and Public Works, March-April, 1944.

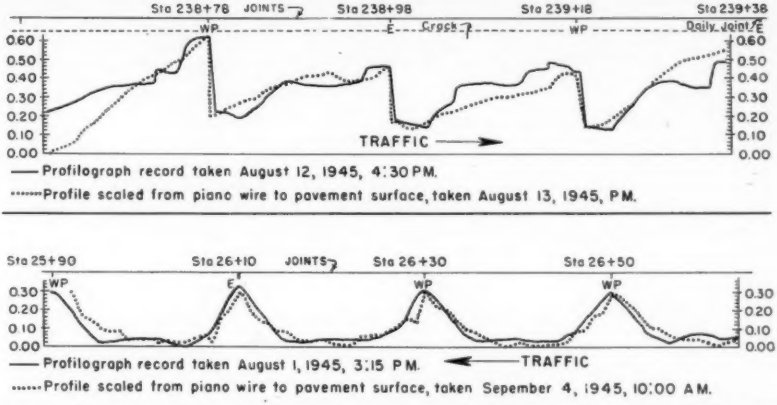

Fig. 1—Relative accuracy of profilograph records compared with profile obtained by stretching a piano wire and scaling offset to pavement surface. Bottom profile represents section of very smooth pavement constructed in 1937. Surrounding country was flooded early in 1938 and pavement curled soon after. At present, pavement has no cracks and is in excellent condition except for curl. Profiles taken 30 in. west of longitudinal joint

had not changed appreciably by the end of one week (Fig. 2C). However, the fourth profile (recorded in the early morning on July 10 when the pavement was 32 days old) shows a definite curling of all slabs.

It is evident that this curl must be due to the combined effects of temperature differential plus moisture. If the curl were due to temperature differences alone, then the fifth profile (Fig. 2E) taken in the afternoon of July 9 should show the slabs convex upward. As the slabs are flat and the surface virtually indistinguishable from the original profile, it is evident that for this pavement, expansion of the upper surface due to temperature only compensates for expansion of the underside caused by moisture.

Effect of soil conditions

Profilograph records can be used to study and analyze many aspects of pavement performance in relation to local conditions. For example, Fig. 3 illustrates types of roughness in relation to subgrade soil type. Fig. 3A shows the marked faulting which developed over a clean sand subgrade, while Fig. 3B and 3C illustrate sections of pavement on the same route where the subgrade is a clayey silt.

Fig. 3D is interesting as it shows a marked contrast in the condition of the pavement due apparently to a grove of trees along the roadside. A closely spaced windbreak of large eucalyptus trees is planted on the property line along the pavement represented by the profile (from the left edge of the sheet to the center, see note on profile). The moisture content of soil under the pavement adjacent to the trees was found to be only 53 percent of that required to fill the voids while the subgrade soil beyond the influence of the trees was 98 percent saturated. The roughness of the pavement is much more

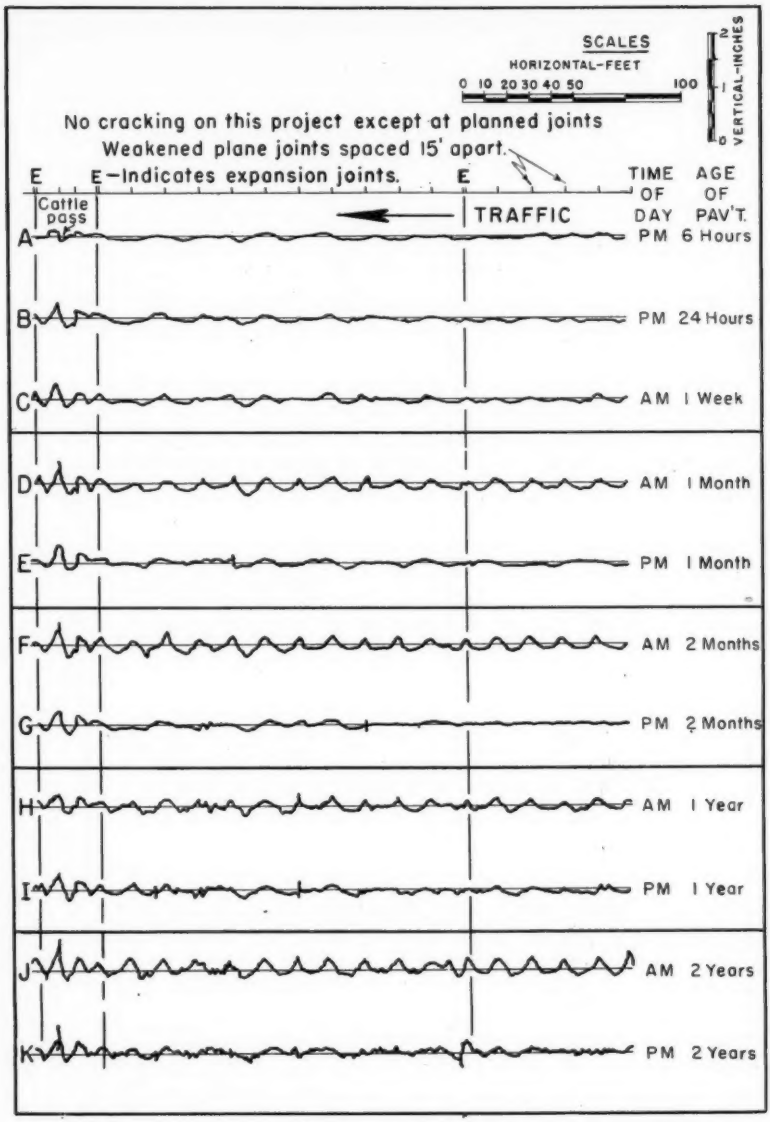

Fig. 2—Profilograms showing development of curling as pavement increases in age

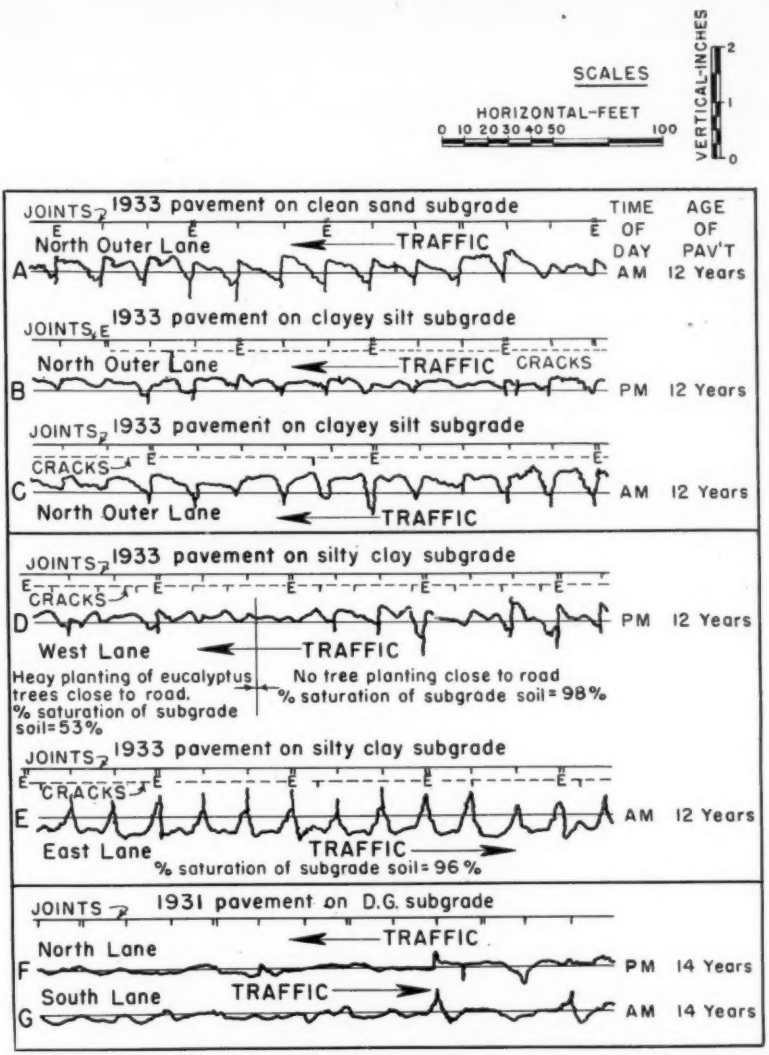

Fig. 3—Profilograms illustrating pavements placed on different types of subgrade soils. Weakened plane joints 20 ft apart. Expansion joints 60 ft apart, except north outer lane C where spacing is 100 ft

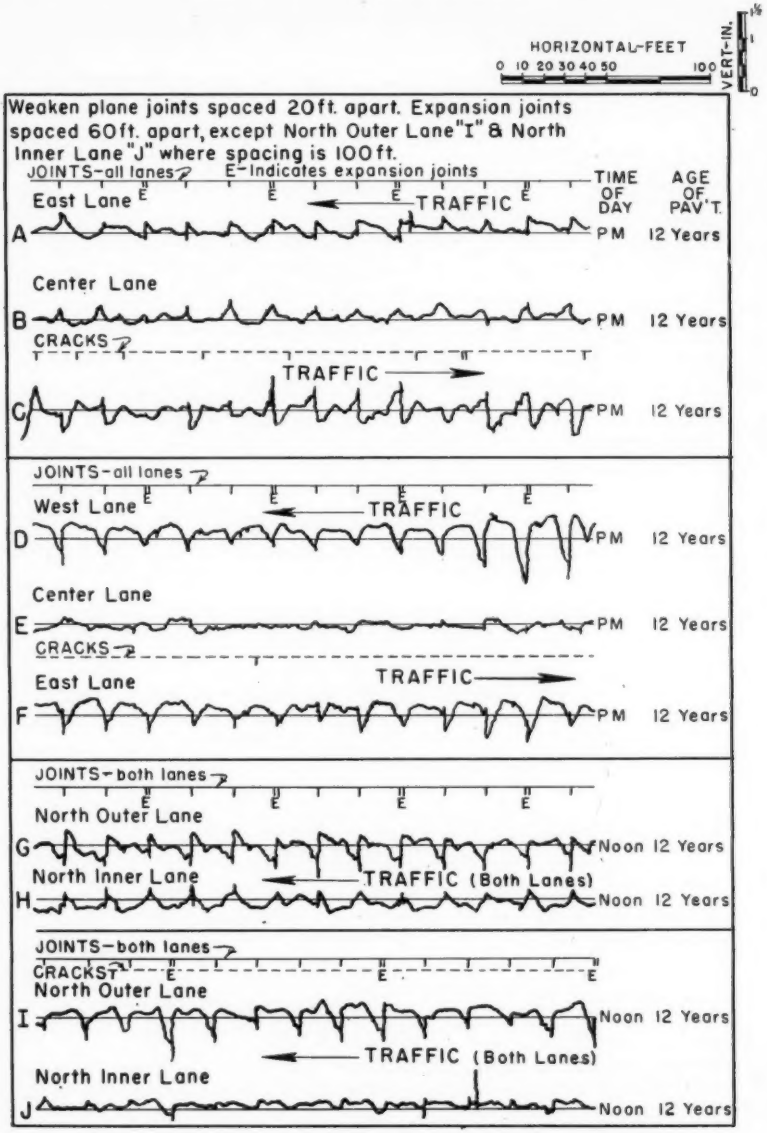

Fig. 4—Profilograms illustrating difference between inner and outer lanes

marked and the faulting at the joints quite apparent where there were no tree roots to draw moisture from beneath the pavement.

Traffic

The effects of traffic are shown clearly by comparing the profilograph trace at inner and outer lanes on multiple lane highways. Fig. 4 shows the marked faulting on the outer lanes of two 3-lane pavements and on one section having four lanes. The lighter traveled inner lane on project Ora-60-A,B,C (Fig. 4B) shows definite evidence of the curled slabs that seem to be a characteristic forerunner to faulting.

This curled condition is not so apparent on LA-4-D (Fig. 4E), and this pavement is unique in California experience. While the outer lanes (Fig. 4D and F) clearly show the effects of traffic, the downward depression at the joints is peculiar to this job. In spite of roughness and depression at the joints this pavement has few transverse cracks. Perhaps cracks occur most frequently when pavement slabs are curled upward and are thus deprived of subgrade support. Fig. 4G to J show the difference between inner and outer northbound lanes of a four-lane highway.

These profilograms indicate that a concrete pavement is far from being inert or static. Fig. 5 and 6 show a magnified profile of single slabs 15 ft long on two different sections of pavement in southern California. Each shows a variation in surface contour between early morning and afternoon. Electronic deflection gages had been installed in these two slabs to measure vertical deflections under passing wheel loads at the ends of the slab, at the center point, and at the quarter point. These deflection measurements are plotted for comparison directly below the slab profiles and clearly indicate that the deflections are greatest when the slab is curled or warped to the greatest extent, which is not particularly surprising. Of greater interest is the fact that the *measured deflection at slab ends is little or no greater than at the centers of the slab when the pavement is flattened down and presumably resting upon the subgrade throughout the entire length of the slab.*

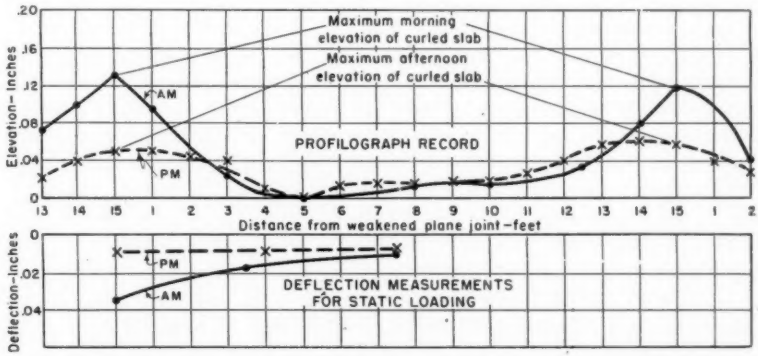

Fig. 5—Comparison of profilogram and deflection of single slab due to static load

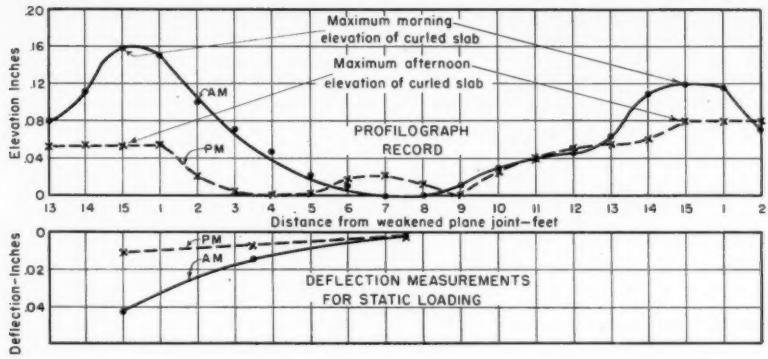

Fig. 6—Comparison of profilogram and deflection of single slab due to static load on another project

WARPING TESTS

To further investigate the factors which influence warping of slabs, a series of test specimens were prepared by cutting thin slices or discs from 8-in. diameter concrete pavement cores with a diamond saw. These discs were dried to constant weight and then placed in a specially designed frame or "spider" (Fig. 7) to measure expansion. The entire assembly was immersed in water and the total expansion measured.

Fig. 8 indicates expansion developed in seven days in thin discs of concrete sawed from cores taken from existing pavements. Discs are 8 in. in diameter and approximately ¼ in. thick. The ordinate on this chart represents expansion developed after the oven-dried discs were immersed in water. Expansion is expressed as a percentage of the diameter. Several days were required in most cases for the expansion to reach equilibrium after which the temperature of the water bath was varied, and the resulting expansion measured. For comparison it is expressed as the percentage of expansion developed due to a rise in temperature from 40 to 130 F.

On about ⅔ of the specimens, (counting from the left side of the figure) expansion caused by moisture exceeds that caused by temperature, while the remaining ⅓ of the specimens (represented by the group on the right side of the graph) show somewhat less expansion due to moisture compared to the effects produced by temperature. Admittedly, the variations in moisture were extreme, as under service conditions it is unlikely that the moisture content of concrete pavements would be reduced to that of the oven-dried specimens at the beginning of the soaking period. The test was conducted to show the response to the effect of moisture. There is no reason to believe that any "autogenic" process is involved in these volume changes. The thermal expansion for the corresponding discs is also shown. Of the 30 samples illustrated, 21 indicated greater expansion due to moisture than to temperature for the range shown (40 to 130 F). In the majority of cases, however, it

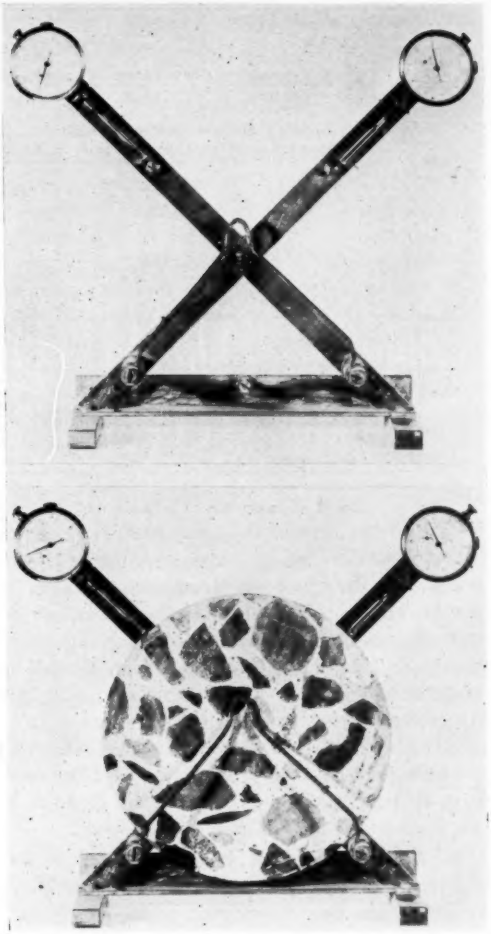

Fig. 7—"Spider" used in measuring thermal and moisture expansion of concrete

appears that under road conditions the expansions due to moisture and to temperature would be approximately the same.

EFFECT OF WEATHER

Volume change of a concrete pavement slab arising from either absorption of moisture or through changes in temperature must usually be nonuniform throughout the depth of the slab. During rain or periods of high humidity the pavement undoubtedly becomes saturated on both upper and lower surfaces and warping stresses are not developed. In summer the upper surface

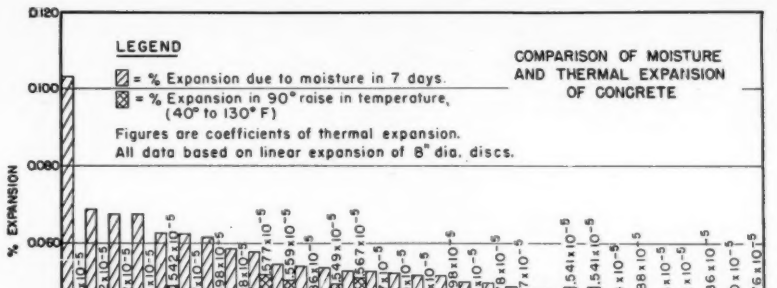
LEGEND
= % Expansion due to moisture in 7 days.
= % Expansion in 90° raise in temperature, (40° to 130° F)
Figures are coefficients of thermal expansion.
All data based on linear expansion of 8" dia. discs.
COMPARISON OF MOISTURE AND THERMAL EXPANSION OF CONCRETE
% EXPANSION
0.120
0.100
0.080
0.060
0.040
0.020
Sample No

Fig. 8—Comparison of moisture and thermal expansion of concrete

of the pavement becomes dry and during the afternoon, at least, the surface of the pavement may reach temperatures equal to or greater than local atmospheric temperatures. The surface cools off rapidly at night, however, and the pavement will be curled to its greatest extent when the surface of the pavement is dry and cool and the underside is relatively warm and moist. Most of the rainfall in California occurs during winter months, and during this period curling or warping is less noticeable as the pavements tend to be more or less uniformly cool and damp throughout. Summer rains in California are infrequent, and as a result the greatest warping (causing the ends of the slabs to be lifted above the subgrade) occurs during early morning hours in late spring or early fall months.

While some pavements gave little or no evidence of daily warping and movement, the majority of concrete pavements in California do show such evidence, and this observation seems to warrant some speculation concerning the applicability of Westergaard's coefficient of subgrade reaction. Engineers have paid much attention to subgrade support for concrete pavements at or near the planned joints in the pavement, but if it is recognized that for a considerable portion of the time rigid pavement slabs do not rest on the subgrade for several feet either side of the joints, it seems pertinent to ask whether Westergaard's K is a significant index for the design of such pavement.

SUBGRADE TREATMENT

Most highway engineers experienced in maintenance work will be aware of the great difficulty, if not virtual impossibility, of completely sealing a

pavement against the entrance of water into the subgrade. The space developed between a curled slab and subgrade provides room for the accumulation of water.

To combat the tendency for subgrade soils to be pumped out from beneath the slabs under the action of traffic, California has adopted the practice of hardening the subgrade with cement and covering with a bituminous membrane in an attempt to create an erosion resistant layer immediately beneath the concrete. The first installation of this type is now four years old, and so far no evidence of mud pumping has been observed. However, it will require several more years before adequate and conclusive proof is obtainable.

At the same time that special subgrade treatments were adopted, California also abandoned the use of expansion joints except at bridge abutments, etc. The standard concrete pavement now has an 8 in. uniform thickness, contraction or dummy joints at 15-ft intervals and no expansion joints. While it is evident that slabs continue to follow a daily cycle of curling with elevated joints in the early morning and there is evidence that moisture does collect beneath the slabs, there is no evidence of mud pumping or faulting over the treated subgrades.

CONTINUOUS REINFORCEMENT

Another means proposed for counteracting the ill effects of expansion of concrete pavements is continuous reinforcement. Several experimental sections have been constructed, one in California on the route between Sacramento and San Francisco constructed in 1949. This experimental section is one mile long on which a concrete pavement of uniform 8 in. thickness was placed over 16 in. of imported granular material of which the top 4 in. was "stabilized" or treated with cement. Continuous steel bars were placed at 4 and 5-in. centers, and SR-4 strain gages were attached to the reinforcing steel at a number of points on which readings have been taken periodically. Detailed data were not available to the author at the time this paper was prepared, but it appears that all gage readings indicate that the steel is in tension throughout.

Aside from frequent fine cracks the section is in excellent condition and may reasonably be expected to give long and satisfactory service. So far as present evidence is concerned, however, the same may be said of the adjoining unreinforced sections. Under the defense mobilization program it seems unlikely that reinforcing steel will be available for pavement construction or for any other purpose where substitute methods or materials can be used. Therefore, any question concerning the merits or demerits of continuous reinforcement in concrete highway pavements appears to be somewhat academic at the present time.

CONCLUSION

There is evidence that poor performance of a concrete pavement may be due to any one or a combination of factors among which the following are

the most prevalent or probable causes: pavement joints at close intervals, high percentage of water in the subgrade soil, high percentage and frequency of heavy axle loads, or expansion and contraction of pavement due to variations in moisture content and nonuniform temperature.

The problem may be remedied by eliminating expansion joints and placing contraction joints as far apart as possible.

No practical or economical method has been developed that will prevent moisture from accumulating in the soil underlying a pavement.

There appears to be no ground for assuming that vehicle axle loads can be reduced or even controlled except within wide limits.

Any method or treatment that will reduce the volume change characteristics of portland cement concrete will be a major contribution to the more successful performance of concrete pavements.

Title No. 47-53

Identification of Dehydrated Gypsum in Portland Cement*

By J. L. GILLILAND†

SYNOPSIS

False set in portland cement is usually attributed to dehydrated gypsum. Inasmuch as hemihydrate is much more soluble than gypsum, the concentration of sulfate ion in the liquid phase of cement pastes can be used to identify cements in which dehydrated gypsum is likely to cause trouble. The solubility test correlates with the penetration test used to detect stiffening.

While X-ray diffraction and differential thermal methods can be used to identify various forms of calcium sulfate in high concentrations, they are not applicable to the study of calcium sulfate in portland cement.

INTRODUCTION

Dehydrated gypsum is the primary cause of false set in portland cement, according to most investigators who have studied the problem. The various theories on false set, and on the mechanism by which dehydrated gypsum causes false set were reviewed in a recent JOURNAL paper.[1] From that study, it was concluded that most of the dehydrated gypsum is due to high grinding temperatures, and to the storage of hot cement.

CALCIUM SULFATES

Kelley, Southard and Anderson[2] state that the existence of two varieties of dihydrate, at least three varieties of hemihydrate, and four varieties of anhydrite has been postulated. The six forms of calcium sulfate which those authors considered as being unique and reproducible are:

1. $CaSO_4 \cdot 2H_2O$, calcium sulfate dihydrate, commonly known as gypsum.
2. $CaSO_4 \cdot \frac{1}{2}H_2O(\alpha)$, one of the calcium sulfate hemihydrates, commonly known as plaster of paris or gypsum plaster.
3. $CaSO_4 \cdot \frac{1}{2}H_2O(\beta)$, a more unstable form of plaster of paris.
4. $CaSO_4$(Sol.α), the soluble anhydrite prepared from α-hemihydrate.
5. $CaSO_4$(Sol.β), the soluble anhydrite prepared from β-hemihydrate.
6. $CaSO_4$(Insol.), the insoluble anhydrite.

All of these forms, except the first, are classed as "dehydrated gypsum." However, most false set is believed to be "plaster set," due to the hydration of hemihydrate. The soluble anhydrites are rapidly converted to hemi-

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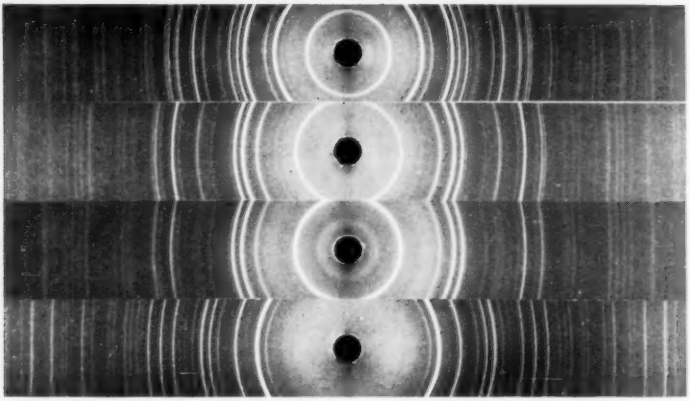

Fig. 1—X-ray diffraction patterns of various forms of calcium sulfate

a—Gypsum (Baker's C.P. $CaSO_4 \cdot 2H_2O$) dissolved in water, then evaporated almost to dryness, followed by air drying. *b*—Plaster of paris ($CaSO_4 \cdot \frac{1}{2}H_2O$). Prepared by heating gypsum for 24 hours at 125 C. *c*—Soluble anhydrite. Prepared by heating gypsum in differential thermal apparatus to 300 C and sealing while hot in thin-walled Pyrex tube. *d*—Natural anhydrite ($CaSO_4$)

hydrates, after which they would perform in the same manner. For the cement technologist, the principal problem is to identify when hemihydrate (or soluble anhydrite) is present. The insoluble anhydrite has not been shown to cause trouble.

X-RAY DIFFRACTION PATTERNS

It is not easy to distinguish between all of the various modifications of calcium sulfate. However, certain forms can be readily identified.

For example, X-ray diffraction patterns provide an excellent method for differentiating between principal forms, but neither of the two hemihydrates or the two soluble anhydrites can be distinguished from each other. The differences between gypsum, hemihydrate and insoluble anhydrite (Fig. 1) are apparent even to an untrained observer. The difference between hemihydrate and soluble anhydrite is more obscure, but sufficient for a trained observer to identify soluble anhydrite as an individual compound. However, soluble anhydrite is so reactive that it does not assume the practical importance of the other modifications. The sample photographed for Fig. 1 was sealed in a glass capillary to prevent moisture reabsorption. Judging from the method of preparation, it seems probable that the hemihydrate and soluble anhydrite in Fig. 1 are the β modification.

The effect of heat is illustrated in Fig. 2. Note (by comparison with Fig. 1) that the analytical grade calcium sulfate dihydrate used actually contained a considerable amount of hemihydrate, and that the dihydrate was completely converted to hemihydrate by heating to 128 C (262 F). No additional change is apparent on heating to 200 C (392 F). At 300 C (572 F), however, a change

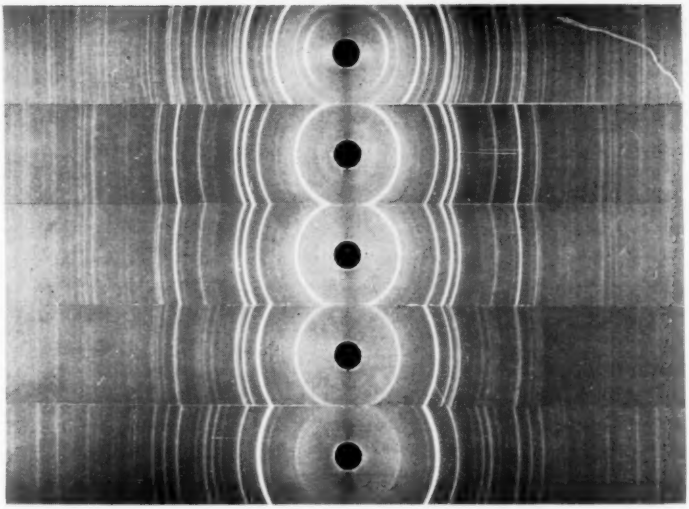

Fig. 2—X-ray diffraction patterns showing effect of heat on calcium sulfate
a—Gypsum (Mallinkrodt analytical grade) as received. b—Same as a, heated to 128 C for 18 hours. c—Same as a, heated to 200 C for 24 hours. d—Same as a, heated to 300 C for 24 hours. e—Same as a, heated to 350 C for 24 hours.

begins, and anhydrite appears faintly. At 350 C (662 F), the X-ray pattern indicates that calcium sulfate has been completely converted to insoluble anhydrite.

Unfortunately, when portland cements are examined by X-ray diffraction, the concentration of calcium sulfate is too low to permit identification of its form.

DIFFERENTIAL THERMAL ANALYSIS

Differential thermal analysis has been used to study thermal reactions of carbonates, clays, zeolites, etc. The use of this technique for the investigation of hydrated portland cements has been described by Kalousek, Davis and Schmertz.[3] As a further aid to identifying different forms of calcium sulfate in commercial cements, thermal analyses were applied to some of the materials used in this study.

The furnace was nichrome wound and had a rate of temperature change of 10.9 C (19.6 F) per minute. The test sample, approximately 0.4 g, was placed in one well of the Inconel sample block, surrounding the tip of one junction of a differential chromel-alumel thermocouple. An adjacent well, containing the other junction of the differential thermocouple, was filled with ignited Al_2O_3 as an inert reference. As the furnace was heated at a constant rate, any temperature differences that developed between the unknown and the inert reference sample caused a deflection of a galvanometer with the deflection being recorded on photographic paper. The temperature of the sample under investigation was obtained by use of another thermo-couple imbedded in inert Al_2O_3 and connected to a galvanometer and potentiometer in such a manner that the temperature could be recorded on the same record at 100 degree intervals.

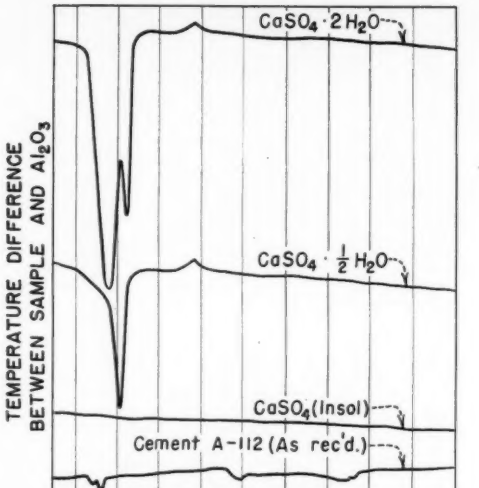

Fig. 3—Differential thermal curves

Typical differential thermal curves are shown in Fig. 3. Gypsum shows an endothermic reaction starting at 100 C (212 F), with a peak at 190 C (374 F), representing the loss of 1½ molecules of water to form hemihydrate. Another endothermic reaction, starting at 210 C (410 F) with a peak at 225 C (437 F), denotes the loss of ½ molecule of water to form soluble anhydrite. An exothermic reaction starting at 320 C (608 F) with a peak at 380 C (716 F) appears to mark the conversion of soluble anhydrite to insoluble anhydrite.

The curve for hemihydrate is similar to the gypsum curve, except that the first strong endothermic reaction at 100-190 C (212-374 F) is absent. This sample of hemihydrate was prepared by heating gypsum in one of the sample wells of the differential thermal machine to 180-190 C (356-374 F), cooling the furnace to room temperature and running the sample again in the thermal apparatus by normal procedure. The peak temperature for this sample of hemihydrate occurred at a slightly lower temperature than the corresponding peak in gypsum, probably because some of the hemihydrate was converted into soluble anhydrite during the original heating. This illustrates that the peak occurs at lower temperatures when smaller quantities of the material are present. $CaSO_4$(Insol.), prepared by heating gypsum to above 400 C (752 F), does not show any thermal reactions in the range studied.

Kalousek and co-workers showed a thermal curve (Fig. 7 of reference 3) for one sample of dry cement, prepared by thoroughly blending ground clinker and gypsum. The presence of gypsum is apparent from their curve, as indi-

cated by the characteristic endothermic reactions with peaks at 170 C (338 F) and 190 C (374 F). However, the curve for a commercially ground cement (Fig. 3) has a larger endothermic reaction at 160 C (320 F) than at 140 C (284 F), indicating that most of the calcium sulfate is present as the hemihydrate and only a small amount is present as unconverted gypsum. The thermal curve of cement No. A-112 after heating to 300 C (572 F) indicates that some calcium sulfate is present as the hemihydrate. The calcium sulfate was probably converted to soluble anhydrite during heating, then partially reconverted to hemihydrate when exposed to air. This partial rehydration to hemihydrate is shown by the decreased amplitude of the thermal reaction at 130-135 C (266-275 F) and a shift of the peak to a lower temperature than is indicated for hemihydrate in the other curves of Fig. 3.

SOLUBILITY RELATIONSHIPS

The solubilities in water of $CaSO_4 \cdot 2H_2O$, $CaSO_4 \cdot \frac{1}{2}H_2O(\alpha)$, and $CaSO_4$ (Insol.) are shown in Fig. 4. The values for $CaSO_4 \cdot 2H_2O$ and for $CaSO_4$ (Insol.) were reported by Posnjak.[4] The $CaSO_4$ (Insol.) was prepared by heating gypsum for a few hours at 500 C (932 F). The values for $CaSO_4 \cdot \frac{1}{2}H_2O(\alpha)$ were reported by Riddell.[5] In addition, Riddell reported results for $CaSO_4 \cdot \frac{1}{2}H_2O(\beta)$ and for $CaSO_4(\beta\text{-sol.})$, but Kelley and Southard,[2] as well as Posnjak believe that these values represent a transient condition. Riddell's results show the solubility of β-hemihydrate to be slightly greater than the solubility of α-hemihydrate, confirming the greater stability of the α-modification.

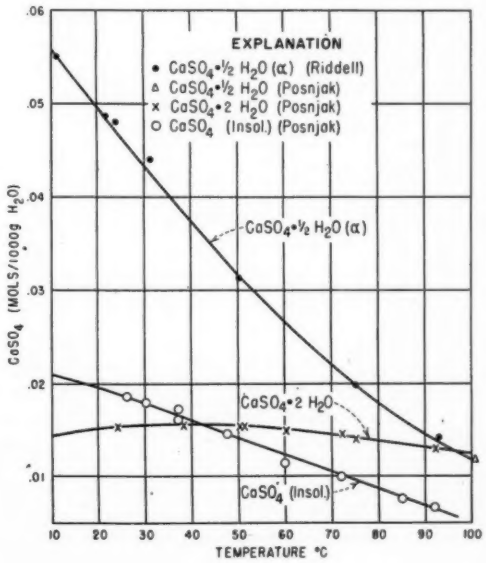

Fig. 4—Solubilities of the calcium sulfates

The solubility curve for hemihydrate represents a metastable condition. Since this solution is supersaturated with respect to dihydrate, at room temperature gypsum precipitates until the solubility shown by the gypsum curve is reached. However, the conversion of hemihydrate to dihydrate is not too rapid to prevent the use of solubility data for identifying hemihydrate in portland cement, providing the extraction is made quickly.

Some time ago, Hansen[6] conducted an unpublished study on the relationship between the composition of the liquid phase of cement paste and premature stiffening or false set characteristics. Hansen, recognizing the effect of alkalies on gypsum solubility, made a series of five water extractions on the same sample of cement. In that way he could determine the sulfate solubility after the alkalies, which are readily soluble, had been removed. He concluded that by the third extract, the alkali concentration was sufficiently low to have little effect on the sulfate solubility. He concluded that dehydrated gypsum probably was present if the SO_3 content of the third successive extract was considerably above 0.1 g per 100 ml.

GYPSUM SATURATION TEST

It has been well established by Roller[7] and Kalousek, Jumper and Tregoning,[8] that the liquid phase of cement pastes during the early stages of hydration is essentially a mixture of the sulfates and hydroxides of calcium, sodium and potassium.

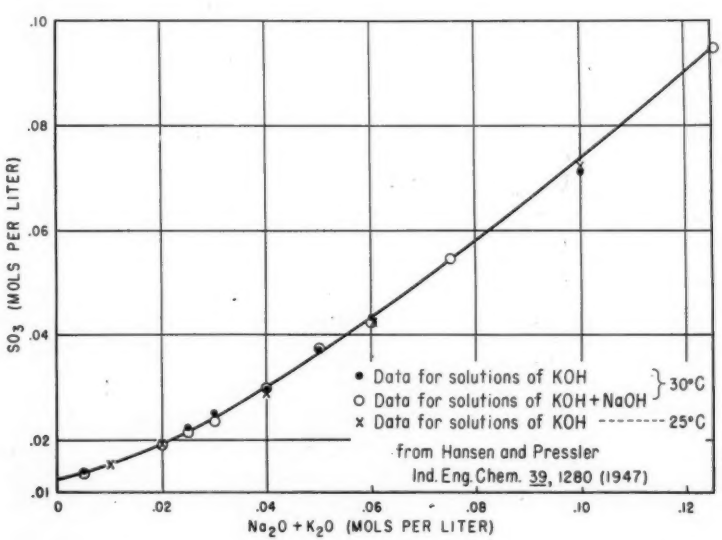

Fig. 5—Solubility of gypsum in saturated calcium hydroxide solutions containing alkali hydroxides

Hansen and Pressler[9] determined the solubility of gypsum in saturated calcium hydroxide solutions containing various amounts of sodium and potassium hydroxides. The solubility of gypsum under these conditions is shown in Fig. 5, which includes only that portion of their data falling within the concentration range normally encountered in portland cement pastes. Fig. 5 shows that the solubility of gypsum is a function of the alkali concentration if the solution is saturated with calcium hydroxide. They used these data to establish whether the liquid phases of cement pastes were supersaturated with respect to gypsum. They concluded that where a supersaturated condition was found, it was due either to greater solubility of hemihydrate, or to extreme fineness of the gypsum resulting from grinding.

In the X-ray diffraction studies described earlier, it was found almost impossible to prevent gypsum from dehydrating during fine grinding operations. Therefore, it seems logical to assume that high solubility values of finely ground gypsum reflect the solubility of the hemihydrate. The acceptance of this assumption permits the identification of hemihydrate on the basis of the composition of the extract. Insoluble anhydrite cannot be identified from the composition of the extract, since its solubility is close to that of gypsum.

The solubility curve of Fig. 5 was applied to analytical data previously reported by several investigators. For the most part, these data are concerned with the composition of the liquid phase of cement pastes, but the work of Jones[10] is on systems containing calcium hydroxide, sulfate and sulfoaluminate as solid phases. The values are shown in Table 1. Note that the actual SO_3 concentrations in these liquid phases are approximately equal, or slightly less than the saturation SO_3 value from the curve. No supersaturation to gypsum is shown. This is not unexpected, since the cements in Table 1

TABLE 1—COMPOSITION OF EXTRACTS FROM CEMENT PASTES

Identification No.*	Cement	Na_2O+K_2O, mols per l	SO_3, mols per l	Sat. SO_3, mols per l	Investigator
1	I	0.0671	0.0490	0.0486	Roller [7]
2	II	0.1304	0.0911	0.0970	Roller [7]
3	III	0.1867	0.1357	0.1400	Roller [7]
4	S-A	0.1166	0.0771	0.0873	Roller [7]
5	16839 (3.5% gypsum)	0.0723	0.0273	0.0278	Lerch [11]
6	16839 (5.0% gypsum)	0.0708	0.0276	0.0274	Lerch [11]
7		0.0000	0.0123	0.0125	Jones [10]
8		0.0891	0.0660	0.0652	Jones [10]
9		0.1251	0.0939	0.0900	Jones [10]
10	11 (4% gypsum)	0.0605	0.0365	0.0438	Forsen [12]
11	5 (1.75% SO_3 as gypsum)	0.1388	0.0970	0.1055	Kalousek, et al [8]
12	8 (1.75% SO_3 as gypsum)	0.0501	0.0276	0.0366	Kalousek, et al [8]
13	9 (1.75% SO_3 as gypsum)	0.0438	0.0260	0.0325	Kalousek, et al [8]

*1, 2 and 3	Commercial slow-setting cements.
4	Slightly seasoned clinker containing 2 percent added gypsum.
5 and 6	Clinker and gypsum ground in a laboratory mill.
7	Solid phases, $CaSO_4 \cdot 2H_2O$, $Ca(OH)_2$ and $C_3A \cdot 3CaSO_4 \cdot 32H_2O$
8	Same as No. 7. 1 percent KOH in liquid phase.
9	Same as No. 7. 1 percent $NaOH$ in liquid phase.
10	Commercial cement to which was admixed 4 percent gypsum.
11, 12 and 13	Type II cements to which gypsum was added.

are laboratory grinds or blends, and have not been subjected to commercial grinding mill temperatures.

The gypsum saturation test, adapted from the work of Hansen and Pressler, was applied to a number of cements which had been checked for false set characteristics by means of the penetration test described by Blanks and Gilliland.[1] The cement paste was vigorously agitated for a short period, then the solid phase was quickly filtered out before the more stable dihydrate could crystallize from the extract solution. Because of the speed with which the test was performed, it was thought that the solubility values reflected the condition of calcium sulfate in the cement rather than equilibrium conditions in the presence of gypsum.

The paste is mixed in a Waring Blendor, which has a blade rotating at 15,000 rpm. Distilled water (125 ml) is placed in the Blendor, then 250 g of cement added as the blade rotates. The paste is mixed for 2 minutes, then rapidly transferred to a Buchner funnel, and filtered under suction through a No. 40 Whatman paper, or equivalent. Usually about 60 ml of extract are recovered. Part of this extract (25 ml) is pipetted into a beaker and used for the determination of SO_3 by precipitation with barium chloride. A 10-ml portion of the original extract is diluted and used for the determination of alkalies by flame photometer method. From these alkali values, the gypsum saturation value was computed by means of Fig. 5.

The results are shown in Table 2. It was found that the cements meeting the penetration test showed only a small amount of SO_3 in excess of the gypsum saturation curve. Conversely, those cements which failed the penetration test were found to be supersaturated with respect to gypsum, indicating the presence of significant amounts of hemihydrate.

Later, Hansen reminded the author that high amounts of SO_3 might appear in the liquid phase if the clinker contained a considerable amount of alkali sulfate. In this case, the degree of supersaturation to gypsum could not be judged if the extract was not saturated with calcium hydroxide. In more recent tests, calcium in the paste extract has been determined and compared with a saturation curve for calcium.

TABLE 2—COMPARISON OF FALSE SET TO GYPSUM SUPERSATURATION

Cement No.	Penetration test			Gypsum saturation test			
	Water, percent	½ min., mm	5 min., mm	Na_2O+K_2O, mols per l	SO_3, mols per l	Sat. SO_3, mols per l	Excess SO_3, mols per l
9938	27.2	37	8	0.0297	0.0463	0.0241	0.0222
9939	27.2	34	2	0.0271	0.0394	0.0227	0.0167
9940	29.6	37	0	0.0136	0.0433	0.0166	0.0267
9941	29.6	36	3	0.0528	0.0625	0.0383	0.0242
9957	27.2	33	4	0.0144	0.0455	0.0169	0.0286
9971	27.0	36	2	0.0270	0.0501	0.0227	0.0276
9976	28.0	37	9	0.0207	0.0307	0.0195	0.0112
A-111	25.6	33	0	0.0107	0.0412	0.0156	0.0256
A-112	28.6	37	15	0.0773	0.0626	0.0562	0.0064
A-215	28.0	33	17	0.0472	0.0424	0.0346	0.0078
A-110	28.4	35	5	0.0150	0.0393	0.0171	0.0222
A-102	27.8	37	29	0.0179	0.0205	0.0183	0.0022
A-457	30.0	38	0	0.0413	0.0495	0.0308	0.0187
7488	27.2	33	15	0.0736	0.0658	0.0535	0.0123

TABLE 3—PENETRATION TESTS AND EXTRACT COMPOSITIONS FOR HEATED CEMENTS

Cement No.	Heated 24 hrs, °C	Penetration test			Na_2O+K_2O, mols per l	SO_3, mols per l	Sat. SO_3, mols per l	Excess SO_3, mols per l
		Water, percent	½ min., mm	5 min., mm				
9939	...	27.2	34	2	0.0271	0.0294	0.0227	0.0167
	128	....	..	..	0.0250	0.0510	0.0216	0.0294
	200	28.0	35	1	0.0230	0.0543	0.0206	0.0337
	300	26.0	38	0	0.0240	0.0543	0.0211	0.0332
	350	27.6	37	21	0.0286	0.0248	0.0235	0.0013
	650	29.2	37	30	0.0343	0.0202	0.0268	−0.0066
9940	...	29.6	37	0	0.0136	0.0433	0.0166	0.0267
	128	....	..	..	0.0138	0.0461	0.0166	0.0295
	350	27.0	38	31	0.0140	0.0364	0.0167	0.0197
A-112	...	28.6	37	15	0.0773	0.0626	0.0562	0.0064
	128	29.0	35	0	0.0739	0.0906	0.0537	0.0369
	200	33.0	33	3	0.0733	0.0741	0.0532	0.0259
	300	28.0	37	0	0.0707	0.0946	0.0514	0.0432
	350	28.0	34	26	0.0758	0.0717	0.0551	0.0168
	650	28.2	36	28	0.0823	0.0533	0.0600	−0.0037
A-215	...	28.0	33	17	0.0472	0.0424	0.0346	0.0078
	128	27.2	33	0	0.0468	0.0689	0.0344	0.0345

DEHYDRATING GYPSUM IN CEMENT BY HEATING

Brandenburg[13] concluded that stiffening could be induced in any well-made cement by raising the finish-end mill temperatures to 110 C (230 F) or above. Since this would be expected to dehydrate the gypsum, the effect of heating the cement on the gypsum saturation test was studied. Four cements, two of which passed the penetration test, were heated overnight at 128 C (262 F). This resulted in all four cements showing severe stiffening in the penetration test, and high supersaturation with respect to gypsum in the liquid phase. Two of the cements, No. 9939 and A-112, were then heated for 24 hours at 200 C (392 F), 300 C (572 F), 350 C (662 F) and 650 C (1202 F). The extract compositions are plotted on Fig. 6 and are shown with the penetration test data in Table 3. The most interesting features of this table are the appearance of stiffening in all cements heated to 128 C and its disappearance on heating to 350 C. This is in agreement with the changes in setting which Whitworth[14] noted were due to heating. Returning to Fig. 2, note that the appearance of stiffening coincides with the formation of hemihydrate from gypsum, and that the conversion to insoluble anhydrite accompanies the disappearance of stiffening. The gypsum saturation test identifies the appearance of the hemihydrate, but does not show the expected decrease in SO_3 concentration in all cases where anhydrite is formed.

FALSE SET DUE TO AERATION

Since small amounts of sodium carbonate are known to induce a rigid set, Bogue[15] theorized that alkali carbonates may react with $Ca(OH)_2$, produced by the rapid hydrolysis of C_3S, to precipitate $CaCO_3$, and cause stiffening. In actual practice, carbonation of cement alkalies is attributed to aeration in a humid atmosphere. Because of this theory, the behavior of a high-alkali cement in the gypsum saturation test is of considerable interest. The selected

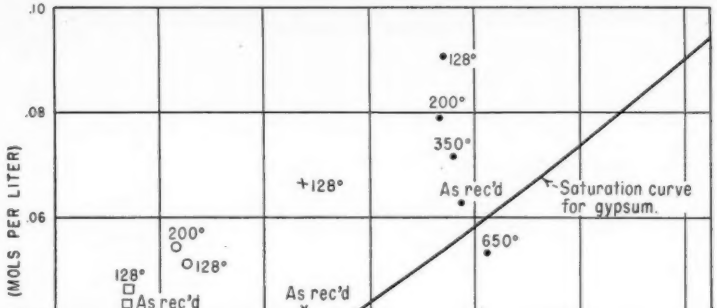

Fig. 6—Composition of extracts from heated cements

cement initially met the requirements of the penetration test, but, as shown in Table 4, false set was induced by aeration for 10 days in a 50 percent relative humidity room. This characteristic disappeared, however, when the cement was heated to 350 C (662 F), and did not reappear on subsequent aeration. Since the CO_2 of Na_2CO_3 and K_2CO_3 is not driven off at this temperature, it must be concluded that any alkali carbonates formed during aeration are still present after heating. The gypsum saturation test on this cement shows that the aerated samples give an extract considerably supersaturated with respect to gypsum, indicating the presence of hemihydrate. These results indicate that some cases of false set attributed to carbonation of alkalies may actually be due to the condition of the calcium sulfate, as influenced by aeration.

SUMMARY AND CONCLUSIONS

Although the principal forms of calcium sulfate can be distinguished from

TABLE 4—FALSE SET STUDIES ON A HIGH-ALKALI CEMENT (1.16 PERCENT Na_2O, 0.04 PERCENT K_2O)

Treatment	Penetration test		Gypsum saturation test			
	½ min., mm	5 min., mm	Na_2O+K_2O, mols per l	SO_3, mols per l	Sat. SO_3, mols per l	Excess SO_3, mols per l
As received	33	15	0.0736	0.0658	0.0535	0.0123
Aerated 10 days at 50 percent R.H.	36	0	0.0733	0.0875	0.0532	0.0343
Heated to 350 C for 24 hours after aeration	37	20	0.0582	0.0371	0.0421	−0.0050

each other by X-ray diffraction patterns and by differential thermal curves, these methods are not applicable to identifying the various forms of calcium sulfate in portland cements.

Calcium sulfate hemihydrate has a metastable solubility considerably higher than the solubility of gypsum or insoluble anhydrite. Therefore, it is possible to identify hemihydrate by solubility studies if the work is performed rapidly. Since false set in portland cement is attributed to the presence of hemihydrate, results from penetration tests were compared with solubility studies. Generally, it was found that cements showing false set in the penetration test gave liquid extracts supersaturated with respect to gypsum.

Heating at low temperatures has been shown to induce false set in cement. At higher temperatures, the set becomes normal. The gypsum saturation test follows this change, identifying the hemihydrate in cement with false set characteristics.

Aeration induces false set in cement. Formerly, it was considered that the alkalies became carbonated during aeration and precipitated calcium carbonate as soon as the cement was gaged. However, the present tests show that the temperature which restores normal set is well below the temperature at which alkali carbonates are decomposed, but that the form of the gypsum is altered.

ACKNOWLEDGMENT

Assistance was received from several co-workers who conducted portions of the studies described in this paper. In particular, acknowledgment is due to O. J. Glantz for the penetration test data, to N. C. Schieltz for the X-ray diffraction patterns, to T. R. Bartley for conducting the gypsum saturation tests, and to M. E. King for the work on differential thermal analysis.

REFERENCES

1. Blanks, R. F. and Gilliland, J. L., "False Set in Portland Cement," ACI JOURNAL, Mar. 1951, *Proc.* V. 47, p. 517.

2. Kelley, K. K., Southard, J. C. and Anderson, C. T., "Thermodynamic Properties of Gypsum and Its Dehydration Products," United States Bureau of Mines Technical Paper 625, 1941, 73 pp.

3. Kalousek, George L., Davis, Curtis W., Jr. and Schmertz, William E., "An Investigation of Hydrating Cements and Related Hydrous Solids by Differential Thermal Analysis," ACI JOURNAL, June 1949, *Proc.* V. 45, p. 693.

4. Posnjak, E., "The System, $CaSO_4$-H_2O," *American Journal of Science*, V. 5, No. 35a, 1938, p. 247.

5. Riddell, W. C., "Physical Properties of Calcined Gypsum," *Rock Products*, V. 53, No. 5, 1950, p. 68.

6. Hansen, W. C., Personal communication. Apr. 1950.

7. Roller, Paul S., "The Setting of Portland Cement," *Industrial and Engineering Chemistry*, V. 26, 1934, p. 669.

8. Kalousek, George L., Jumper, C. H. and Tregoning, J. J., "Composition and Physical Properties of Aqueous Extracts from Portland Cement Clinker Pastes Containing Added Materials," *Journal of Research*, National Bureau of Standards, V. 30, 1943, p. 215, RP 1530.

9. Hansen, W. C. and Pressler, E. E., "Solubility of $Ca(OH)_2$ and $CaSO_4 \cdot 2H_2O$ in Dilute Alkali Solutions," *Industrial and Engineering Chemistry*, V. 39, 1947, p. 1280.

10. Jones, F. E., "The Quinary System CaO-Al_2O_3-$CaSO_4$-K_2O-H_2O (1 percent KOH) at 25° C," and "The Quinary System CaO-Al_2O_3-$CaSO_4$-Na_2O-H_2O (1 percent $NaOH$) at 25° C," *Journal of Physical Chemistry*, V. 48, 1944, pp. 356-394.

11. Lerch, William, "The Influence of Gypsum on the Hydration and Properties of Portland Cement Pastes," *Proceedings*, ASTM, V. 46, 1946, p. 1252.

12. Forsén, L., "The Chemistry of Retarders and Accelerators," *Symposium on the Chemistry of Cements*, Stockholm, 1938.

13. Brandenburg, H. R., " 'False Set' of Portland Cement," *Cement and Cement Manufacture*, V. 2, 1929, p. 96.

14. Whitworth, Frederick, "Effect of Temperature on the Setting Time of Portland Cement," *Cement and Cement Manufacture*, V. 4, 1931, pp. 1001-2.

15. Bogue, R. H., *The Chemistry of Portland Cement*, Reinhold Publishing Corp., New York, 1947.

Title No. 47-54

Cooling Materials for Mass Concrete*

By H. H. ROBERTS†

SYNOPSIS

To maintain proper temperatures in the 1,500,000 cu yd of concrete in Detroit Dam, several techniques were considered. After an investigation of the heat balance in the materials, it was decided to construct a plant for cooling the aggregate and cement. A brief history of cooling mass concrete precedes the description of the cooling plant for Detroit Dam.

The coarse aggregate is immersed in 35 F water until the aggregate cools to 38 F. It is then drained and screened to remove excess moisture.

Sand and cement are cooled by continuous hollow-flight screw conveyors through which 35 F water is circulated. Heat transfer is through the conveyor surfaces; there is no direct contact with the cooled water.

INTRODUCTION

"Temperature control" as applied to construction of massive concrete dams used to be found chiefly in the glossary of technicians and designers. However, today it occupies a prominent position in the specifications of most mass concrete dams and by degrees the requirements have become more exacting until now it is not uncommon to specify a minimum-maximum placing temperature range of 40-50 F for mass concrete. Specifications do not indicate nor dictate the manner or method of accomplishing this; hence it has become a contractor's problem and as such the contracting indus-

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try is responsible for developing the processes now being employed on many major projects.

It is not the intent of this paper to present in detail the purposes of temperature control. However, it might be well to consider a principal reason for specifying temperature control of mass concrete—to reduce cracking caused by thermal expansion and contraction stresses. When new concrete is placed against rock or old concrete, which both offer unyielding restraint, compressive stresses are developed in the new concrete because the heat of hydration raises the temperature of the concrete. The concrete is still plastic and flows, thus tensile stresses develop as the concrete cools. Similar conditions will occur in large horizontal or vertical surfaces should they cool suddenly relative to the central mass, which offers the restraint. Thus surface cracking or crazing will appear, especially undesirable on the upstream and downstream faces of a dam, whether exposed to water action or not. The maximum temperature differential between new concrete and restraining mass, whether it is foundation rock or old concrete, chiefly determines the degree of stresses induced, and if this differential can be kept small, then less cracking can be expected.

Most specifications include some or all of the following limitations, their purpose being to limit cracking to a minimum.

1. A minimum of 120 hours between successive lifts.
2. The first four lifts above foundation rock or above concrete in place longer than 15 days shall be limited to 2½ ft thickness.
3. The maximum height differential between adjacent blocks shall be 25 ft and should these vertical surfaces be left exposed for periods longer than 30 days then protective insulation shall be applied.
4. Limiting placing temperatures.
5. Forced cooling through embedded piping.

The above controls are effective mainly if a continuous and systematic placing schedule is maintained.

HISTORY OF COOLING PLANTS

Temperature control as originally practiced at Boulder, Grand Coulee, Shasta, and Friant Dams and now at Hungry Horse Dam consisted of embedding pipe coils on the horizontal surfaces of each lift and circulating cold or refrigerated water during and after placement, together with using ice for mixing water. This method has proved successful but obviously has limitations and is not economical nor practicable when considering controlled temperatures between 65-40 F. Another aid in controlling temperature is the addition of pozzolanic material to reduce heat of hydration.

The first major dam where 65-50 F placing temperature was specified was Bull Shoals Dam in Arkansas. It was recognized immediately that ice alone could not satisfy the specifications because of the high ambient temperatures from May through September, and some other approach to the problem had to be used. Precooling of the dry ingredients of the mix appeared to be a

solution. A report by V. L. Peugh and Ivan Tyler prepared in 1934 for Pine Canyon Dam project, Pasadena, Calif., on "Mathematical Theory of Cooling Concrete Aggregates," substantiated the idea that to inundate the coarse material would accomplish the desired results and accordingly this approach was used in calculating preliminary heat balances.

The Bull Shoals plant was constructed utilizing inundated coarse aggregates, contact cooling of the sand, no cement cooling and the addition of 35 F mixing water. It was placed in operation in October, 1948, and to date has cooled materials for approximately 2,200,000 cu yd of concrete well within the specified 65-50 F range. This plant has functioned on a 24-hour day, 6 to 7-day week basis under the highest ambient temperature conditions which in turn made possible the placing of almost 170,000 cu yd in one month, and more than 1,000,000 cu yd from November, 1949, through August, 1950.

For the record, the sand cooling method was a failure. However, this was offset by the efficiency of the inundation treatment of the coarse aggregates. Indisputably the inundation method proved successful at Bull Shoals and no doubt is responsible for the present 50 F maximum placing temperature. Buggs Island Dam in Virginia followed the general features of the Bull Shoals plant and further refinement to the process is being employed at Detroit Dam.

The other method of precooling aggregates is the cold air method having been baptised at Harlan County Dam in Nebraska and since used at Philpott Dam in Virginia and Pine Flat Dam in California.

DETROIT DAM COOLING PLANT

Design criteria

The specifications for Detroit Dam state, in part: "The concrete temperature shall not be more than 50 F nor less than 40 F during placement with the provision that refrigeration equipment shall be utilized during any period until the concrete decreases to not more than 40 F . . . . when measured one hour after placement." Consider this as the first condition. The second condition was determined by study of the local climatic records which disclosed that the maximum river water temperature was 60 F and that the average dew point temperature was 60 F, the latter being significant when considering the problem of cooling cement. Conditions 3, 4 and 5 were contractor electives: The assumed specific heat of all aggregates, sand and cement was 0.22. The plant should have capacity to cool materials to satisfy a 250 cu yd per hour placing rate. The plant should be able to sustain this rate on a 24-hour day, 6 to 7-day week basis. These five conditions constitute the basic criteria of design and must be recognized before the design of any plant can be started.

The individual amounts of materials to be handled were obtained from a typical mass concrete mix design and for simplification of computations the ingredients were reduced to water equivalent weights.

Table 1 shows the results of the heat balance equation. Examination of Table 1 will show that an approach temperature of 45 F out of the mixers

TABLE 1—THEORETICAL HEAT BALANCE*

Ingredient and specific heat	Weight, lb.	Free moist., lb.	Input temp., °F	Δ T to 45 F	T min, °F	Water equiv. wt.	BTU load to T min.	Δ H to 45 F +	Δ H to 45 F −	Diff. + −
Cement (0.22)	226	0	150	105	60	50	4500	750		
6 x 3-in. (0.22)	840	8	65	20	38	193	5210		1350	
3 x 1½-in. (0.22)	780	8	65	20	38	180	4860		1260	
1½ x ¾-in. (0.22)	670	7	65	20	38	154	4160		1078	
¾-in x No. 4 (0.22)	400	4	70	25	38	92	2950		644	
Sand (0.22)	1000	64	70	25	50	284	5680	1420		
Mixing water (1.0)	100	—	60	15	35	100	2500		100	
Heat of mixing								1060		
Totals	4016	91	—	—	—	1053	29860	3230	5332	−5332 +3230 −2102

*For 1 cu yd of mass concrete (0.6 bbl. cement; W/C ratio 0.85; 5.5 percent air)

Adjustment: $\frac{-2102}{1053} = -2.0$ F

Theoretical temperature out of mixer: 45° − 2° = 43 F

Required production per minute: $\frac{250}{60} = 4.2$ cu yd

Theoretical tonnage refrigeration required: $= \frac{29{,}860 \times 4.2}{200} = 626$ TR

Assume loss factor of ± 10 percent $= \frac{625}{0.90} = 697$ TR

was chosen. The 5° difference between the specified 50° maximum and the 45° approach would allow for any pickup caused by delays in transit from the mixers to the forms. To satisfy this 45° approach, 38 F was chosen as the final temperature of the coarse aggregates, 50 F as the final temperature of the sand, 60 F as the final temperature of the cement and 35 F as the final temperature of the added mixing water. The equation did not balance; however, the minus "adjustment factor" was favorable which has resulted in a theoretical temperature out of the mixers between 42 and 43 F.

With the heat balance theoretically satisfying the specifications and the required tonnage of refrigeration being determined, the detailed design of the plant resolved itself into a matter of flow of materials, proper implimentation, structural design, hydraulic and mechanical analysis of the circulating system and design of the electronic control system.

Description of process

Fig. 1 is a schematic plan of aggregate and cement cooling and routing. The various sizes of aggregate are transported from storage bins by belt conveyor to the cooling plant and placed in individual steel tanks filled with 35 F water. The water continues to circulate from the bottom of the tank through the voids between particles and discharges over a weir on the top of the tank until the aggregate reaches a temperature of approximately 38 F. Then the water circulation is stopped and the tanks drained. The aggregate is now ready to be transported by conveyor to the mixing plant. Enroute to the mixing plant the aggregate is passed over vibrating screens where excess surface moisture is removed.

The sand and cement are cooled by screw type contact heat exchangers, earlier used in the food industry, now employed for the first time for sand and cement cooling.

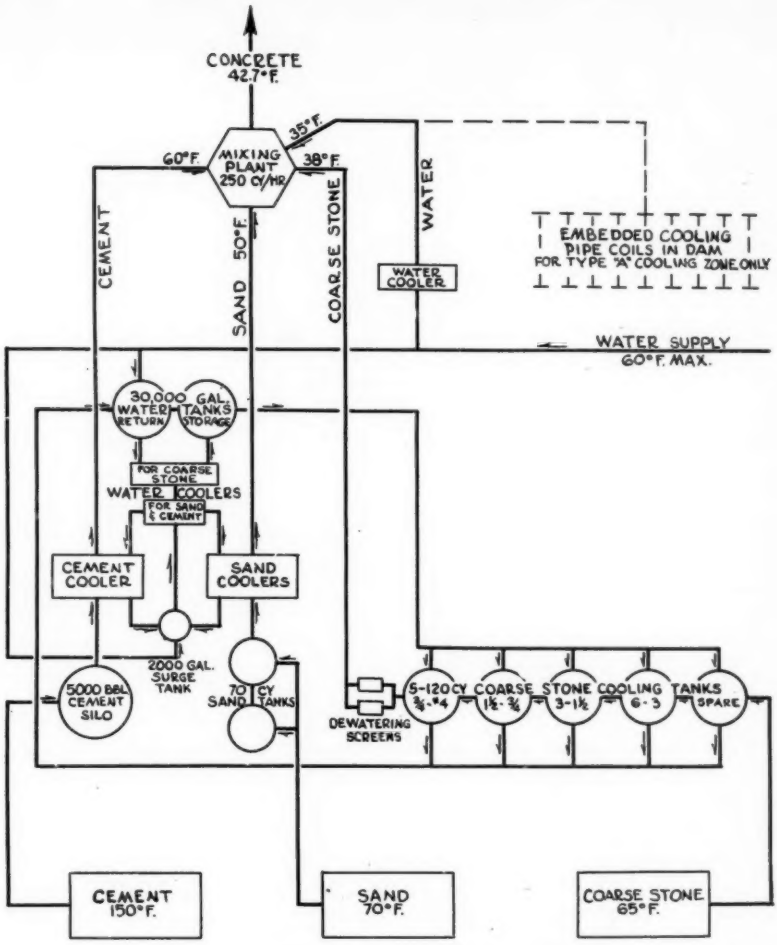
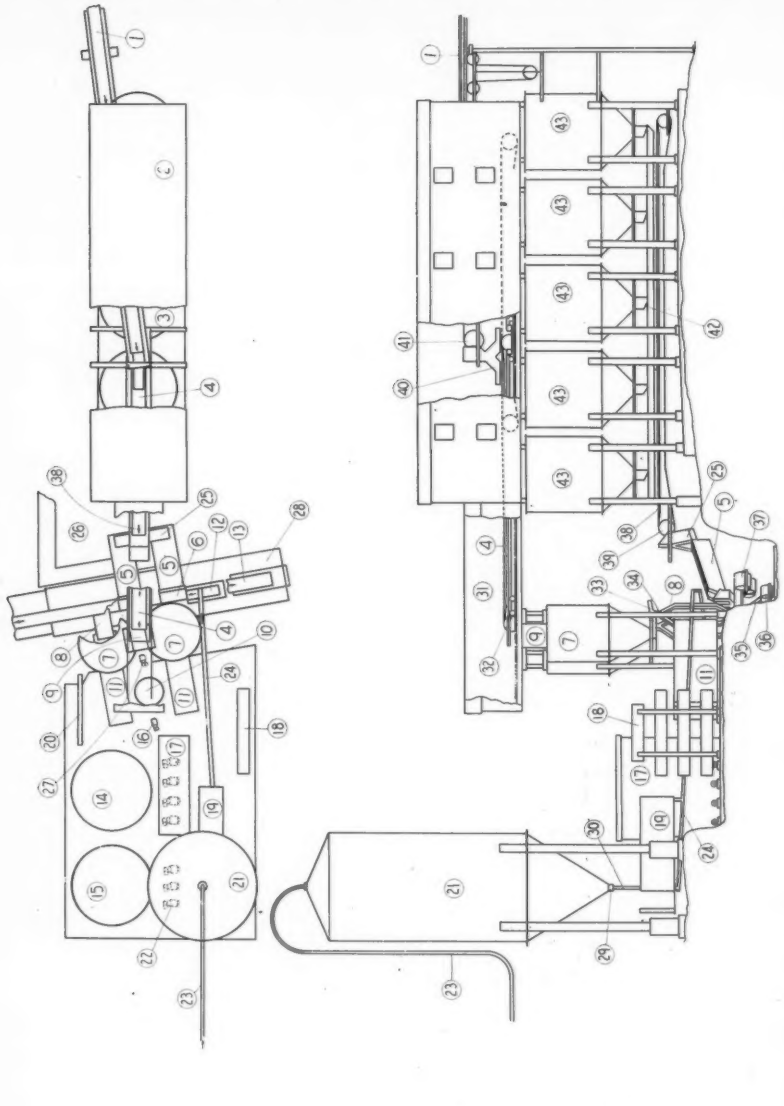


Fig. 1—Schematic plan of aggregate and cement cooling and routing to batching plant



The refrigeration and water cooling equipment is of standard commercial type adapted to the special conditions.

PLANT EQUIPMENT

Refrigeration

The cooling plant equipment for Detroit Dam (Fig. 2) consists of an ammonia compressor plant including ammonia condensers, water coolers with circulating pumps, aggregate cooling tanks, dewatering screens and sand and cement coolers.

To facilitate economical operation during the changing rate of production, six 9 x 9, two-cylinder ammonia compressors of 106 TR (ton-refrigeration) capacity each and one 8 x 8, two-cylinder compressor of 63 TR capacity were selected. Only the smaller and one of the larger units have 50 percent manual capacity control features. The ammonia condensers are of the shell and tube type designed for 700 TR using 285 gpm of river water.

The compressor building is separated from the remainder of the cooling plant because of difficult topography at the damsite and ammonia is piped about 500 ft to the cooling plant proper.

Water cooling

The shell and tube water coolers are arranged in three separate cold water systems, one for coarse aggregate cooling, one for the sand and cement coolers, and one cooler for mixing water supply (Fig. 3). This arrangement keeps silt washed off the coarse aggregate during the cooling process away from the sand and cement coolers and the mixing water.

The water coolers have a rated capacity of 700 TR equal to cooling 2250 gpm from 60 to 35 F. To give sufficient storage for all operating conditions, one 30,000-gal. chilled water tank and one 30,000-gal. return water tank was provided for aggregate cooling water. A 2000-gal. surge tank was installed for the sand and cement coolers.

The water coolers are equipped with automatic float level controls in the ammonia supply lines and anti-freeze-up alarms.

Fig. 2—Plan and elevation of Detroit Dam cooling plant. (1) 36-in. conveyor, 400 ft per min., 700 ton per hour, from sand and aggregate storage; (2) cooling tank housing; (3) control house; (4) 36-in. shuttle conveyor, 400 ft per min., 700 ton per hour; (5) dewatering screens; (6) chute; (7) 70-cu yd sand tank; (8) sand chute; (9) sand splitter chute; (10) 2000-gal. surge tank; (11) sand cooler; (12) 24-in. sand conveyor, 200 ft per min., 150 ton per hour; (13) 36-in. aggregate conveyor, 300 ft per min., 600 ton per hour; (14) 30,000-gal. chilled water storage tank; (15) 30,000-gal. chilled water return tank; (16) 450-gpm pump; (17) power center house, 800-gpm pumps; (18) water cooler; (19) cement cooler; (20) mixing water cooler; (21) 5000-bbl. cement silo; (22) 750-gpm pumps under silo; (23) 8-in. cement line from unloading silos, 250-300 bbl. per hour; (24) cement airslide, 250-300 bbl. per hour; (25) screen feed chutes; (26) waste sump; (27) 600-gpm pump; (28) enclosed bridge to mixing plant with 36-in. aggregate and 24-in. sand conveyors, cement airslide and 2½-in. 35° water line; (29) rotary valve; (30) cement chute and by-pass; (31) shuttle conveyor housing; (32) drive for shuttle conveyor; (33) sand feeders; (34) splitter gates; (35) sand chutes to conveyor; (36) 24-in. sand conveyor drive; (37) 36-in. aggregate conveyor drive; (38) 36-in. aggregate reclaiming conveyor, 300 ft per min., 600 ton per hour; (39) head drive for aggregate reclaiming conveyor; (40) two-way chute to shuttle conveyor; (41) head drive for conveyor from aggregate and sand storage; (42) air-operated roller gates; (43) 120-cu yd aggregate cooling tanks

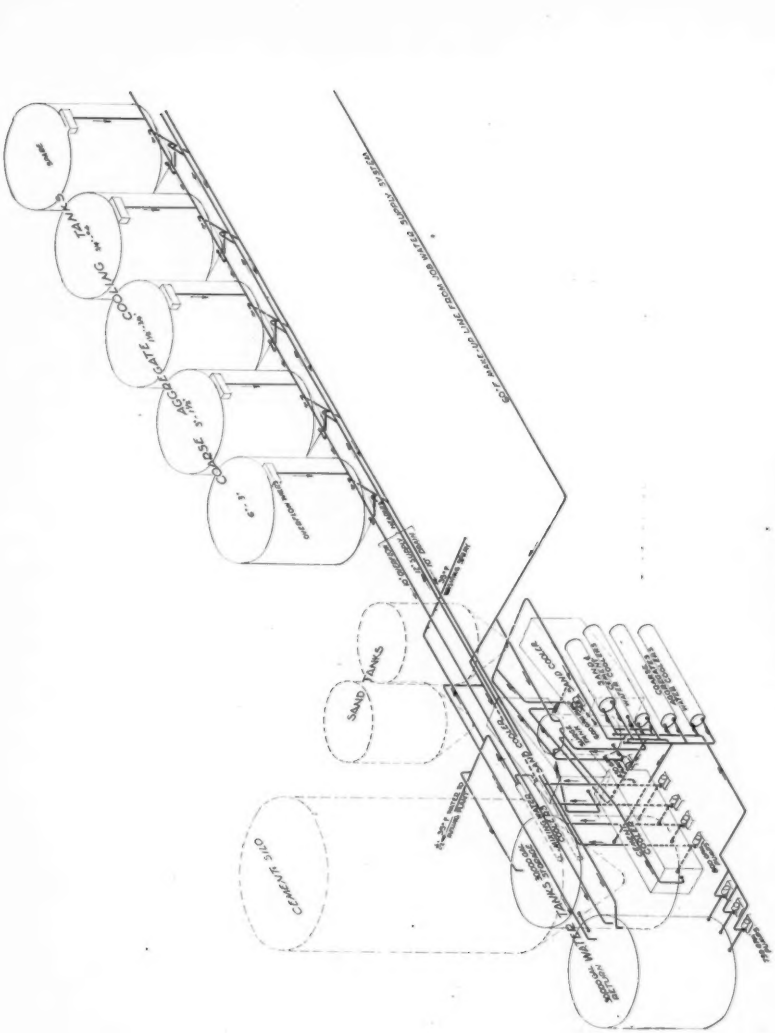


Fig. 3—Water piping diagram

Three 750-gpm pumps circulate water from the return tank through the water coolers to the chilled water tank. From here four 800-gpm pumps working on a common header feed the water through solenoid controlled air-operated valves at each aggregate cooling tank. The gravity flow draining of the tank is controlled by similar valves. Sand and cement coolers are supplied by a 450-gpm pump in closed circuit with the respective water cooler and surge tank.

The mixing water is taken from the make-up water line directly through the cooler.

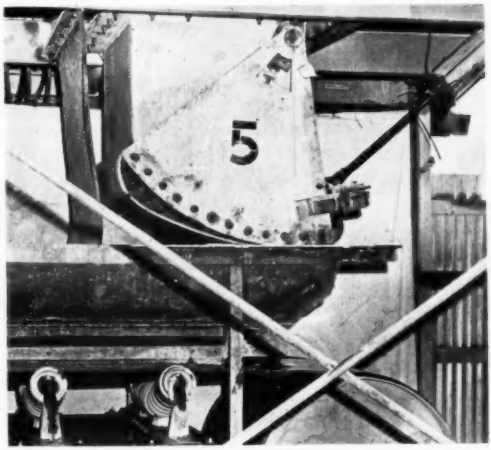


Fig. 4—Watertight roller gates, air-solenoid controlled, cannot empty cooled coarse aggregate tanks on to conveyor before the draining cycle is complete and turn head is positioned at respective bin at mixing plant

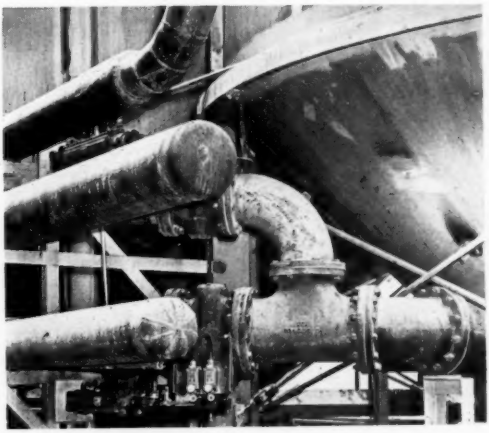


Fig. 5—Fill, circulating and drain water lines have two common inlets at the bottom of each aggregate cooling tank, while circulating water flows over weirs on tops of the tanks to a common return header. Note solenoid controls at bottom for air-operated valves

Fig. 3—Water piping diagram

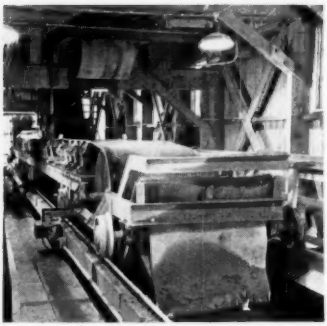


Fig. 6—Aggregate is deposited into cooling tanks by a shuttle conveyor

Aggregate cooling

Aggregate is cooled in five 120-cu yd steel tanks with conical bottoms equipped with job designed and developed watertight roller gates, air-solenoid controlled (Fig. 4). There is one tank for each of the four sizes of aggregate and one spare, which can be used for any type of material.

The fill, circulating and drain water lines have two common inlets at the bottom of each tank, 180° apart, while circulating water flows over weirs on the tops of the tanks to a common return header (Fig. 5). Aggregate is deposited into the tanks by a shuttle conveyor fed by the reclaiming conveyor from aggregate and sand storage bins (Fig. 6).

From the aggregate tanks the cooled rock is transported by a conveyor to a pair of 5 x 10 double-deck dewatering screens discharging onto the conveyor to the mixing plant. The dewatering screens are equipped with air-water jets for washing coated aggregates. All cooling plant conveyors are 36 in. with 600 ton per hour capacity.

Sand and cement cooling

Sand and cement are cooled in two special heat exchangers employing hollow, meshing screws, through which 35 F water is circulated. Sand is fed to the coolers by two vibrating feeders from two sand surge tanks charged by the shuttle conveyor. After cooling, the sand is transported by a 24-in. conveyor to the mixing plant. Cement flows from a 5000-barrel silo through the cement cooler and by an "Airslide" conveyor to the mixing plant. Rated capacity of the sand coolers is 130 ton per hour, the cement cooler 250 barrels per hour.

OPERATION SEQUENCE AND CONTROL

The control system for the plant was designed to meet the specific need for a simple and, as near as possible, foolproof operation. The operator at the turnhead on top of the mixing plant controls the flow of cooled aggregate and the cooling of sand and cement, while the operator at the cooling plant control room is responsible for cooling aggregate and charging the sand surge tanks. The control center for aggregate cooling operations was located

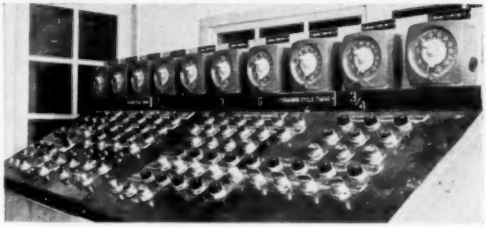


Fig. 7—Electronic control center for aggregate cooling operations

on top of the aggregate cooling tanks to provide the operator with the best possible view of his part of the operation.

The aggregate conveyor to the turnhead, dewatering screens, conveyor under the tanks and tank discharge gates are operated from the electronic control center (Fig. 7) on top of the mixing plant and interlocked for starting in sequence mentioned. This sequence is in turn interlocked with the cooling operation and the turnhead position. No tank discharge gate can be opened before the draining cycle is either complete or manually released by the cooling plant operator, and the turnhead is positioned at the respective bin at the mixing plant. Indicator lights aid the operations.

The sand cooling system has sequence interlocking after a similar pattern, conveyor to mixing plant, sand coolers and vibrating feeders. In addition a manually operated by-pass of the sand coolers can be discharged directly onto the conveyor.

The three circulation pumps for the water coolers have individual start-stop buttons, but two pumps must be running before a solenoid controlled valve in the ammonia supply line opens.

To start the cooling of a batch of any one of the aggregate sizes, the operator pushes the cooling cycle control button for the proper size, which automatically turns on one circulating pump, opens the tank fill valve, releases first interlock for the respective draw-off gate in the reclaiming tunnel, and starts the cooling cycle timer.

Step two is to start the shuttle and reclaiming conveyors in sequence, which releases second interlock to the draw-off gate. By opening the draw-off gate, the shuttle conveyor automatically positions to the respective cooling tank. This operation is delayed by a timer to allow the previous type rock on the conveyors to reach the cooling tank. A timer is also provided to delay the successive opening of draw-off gates for different types of aggregate to allow for the time required to position the shuttle conveyor. The amount of rock dumped into the tank is controlled manually by the operator.

When the timer shuts off the cooling cycle, an interlock releases the draining cycle. The draining time is also controlled by a timer, which in turn gives the green light for the respective discharge gate control at the mixing plant turnhead.

All the operations can be performed manually if desired, and indicator lights aid the operator.

The sand surge tanks are filled by a similar procedure but is independent of the cooling operation.

Normal operating crew for the cooling operation consists of four men: one compressor operator, one cooling plant operator, one oiler and one mechanic. The operator at the mixing plant turnhead belongs to the regular mixing plant crew.

COOLING CYCLE

The plant was designed to cool one batch (120 cu yd) of aggregate in 90 minutes, of which 30 minutes are allowed for circulation of chilled water after the tank is filled with rock. The remainder is for filling, draining and draw-off. Practical experience shows that the cooling time required for coarse rock is somewhat shorter than theoretical studies indicate, while the opposite is the case for the smaller aggregate.

PERFORMANCE

Except for minor details the aggregate cooling equipment has met design expectations.

Due to extremely low bulk weight of the sand (about 80 lb per cu ft) and high moisture content some difficulties have been encountered in operating the sand coolers. The heat exchange properties of the coolers seem to meet design requirements; however, no definite information as to their performance is at present available as cooling of sand has not been necessary during the winter months.

It has not been necessary to use the cement cooler to date; however, it is believed that its performance will meet design expectations.

Considering the developments leading to the final selection of the process and the apparatus for controlling the placing temperature for Detroit Dam it is important to recognize that (1) the heat balance equation and most of the necessary physical data are available in the literature; (2) no novel or unpredictable result is obtained by selection of any of the methods outlined; and (3) the combination of experience gained at Bull Shoals Dam together with the skills of a competent engineering organization supplemented by sound counsel from its advisors has made the design and construction of this plant possible. Some remedial work has already been done, perhaps more will follow, both tending to improve the efficiency of the plant.

ACKNOWLEDGMENT

The following have been most helpful and encouraging in solving the problems attendant to the design, construction and operation of both the Bull Shoals Dam and Detroit Dam cooling plants: M. H. Slocum, project manager, Ozark Dam Constructors; Verne Peugh, general manager, Peter Kiewit and Sons Co., San Francisco; Glen Miller, mechanical engineer, Covina, Calif.; Russel A. Hoffman, general superintendent, Consolidated Builders, Inc.; and Einar Skinnarland, senior structural engineer, Consolidated Builders, Inc.

New data on the physical properties of concretes in which air replaced fine aggregate.

Air Replaces Sand in "No-Fines" Concrete*

By RUDOLPH C. VALORE, Jr.† and WILLIAM C. GREEN†

SYNOPSIS

Concretes containing high-early strength cement, 20 to 30 percent entrained air in place of fine aggregate, siliceous pea gravel and a proprietary resin or detergent air-entraining agent, were made using ordinary rotating tilt-drum mixers. The maximum air contents in mixes having a compressive strength of 500 psi (28 days) were 25 percent (3.3 bag mix) and 29 percent (5.6 bag mix). The ratio of compressive to transverse and bond strengths was about 3. The saturation coefficient and capillarity were much lower, the resistance to freezing and thawing generally higher, and the thermal conductivity (k) 30 to 40 percent lower than for a nonaerated sand-gravel concrete. The drying shrinkage was about the same as for a nonaerated concrete. The compressive strength of all mixes decreased about 100 psi for each percent increase in air content, which was difficult to control.

INTRODUCTION

The advantages of lower density and thermal conductivity of lightweight concretes are often accompanied by economic and technical disadvantages. Whether lower density is obtained by using special aggregates, or by generation of foam or gas requiring special techniques and equipment, the shrinkage upon drying may be so great that the monolithic application of these concretes is inadvisable.[1,2] The work to be described is part of a National Bureau of Standards study of lightweight concretes that might be suitable for construction of monolithic walls and other members of small buildings and other structures. Since the use of easily available (dense) aggregates and ordinary mixing techniques and equipment was considered essential, the entrainment of air during mixing was selected as the means of obtaining lowered density. Conventional fine aggregate was replaced by entrained air.‡ The only structural prerequisite was a minimum compressive strength of 500 psi.

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‡Experimental mixes were made with and without fine aggregate; for mixes in which sand comprised 9 to 15 percent of the total aggregate the water-cement ratios were approximately 12 percent higher than for mixes containing no fine aggregate and having the same cement factors (3.3 and 5.6 bags per cu yd) and air contents (23 to 26 percent).

SCOPE

Preliminary experiments indicated that the lowest cement factor permitting the attainment of 500 psi compressive strength in an "air-gravel" concrete containing pea gravel aggregate, was about 3¼ bags per cu yd. This mix (mix A) was proportioned 1 to 8.6 (by weight). It was found necessary to entrain about 20 percent air in such a mix to provide sufficient cement paste to just fill the spaces between aggregate particles. It was also found that the required compressive strength of 500 psi was not likely to be attained in such a mix containing more than 25 percent air. Since an excess of 3 or 4 percent over the base requirement of 20 percent air was needed to provide even fair workability, the entrained air requirement of mix A was limited to a narrow range. A second mix proportioned 1 to 4.25 (by weight), and having a cement factor of approximately 5.6 bags per cu yd (mix C) was also selected for study. This mix was workable for air contents of 20 to 30 percent; it also appeared that, at equal density, mix C would provide higher strength than mix A. The properties to be determined, and types of specimens cast and tested are listed in Table 1.

The slump requirement was set at 5 ± 2 in., and the air requirement at 25 ± 2 percent for mix A, and 26 ± 3 percent for mix C.

MATERIALS

A high-early strength portland cement meeting Type III requirements of Federal Specification SS-C-192 was used. The aggregate was a siliceous pea gravel obtained from White Marsh, Md.; only the No. 4 to ⅜-in. sieve fraction, in a kiln-dried condition, was used. The physical properties of the aggregate were:

Bulk specific gravity (saturated, surface dry)	2.64
Weight per cu ft (dry, rodded), lb	103.0
Absolute volume per unit dry rodded volume	0.63
Absorption (by weight), percent	0.3

Two air-entraining agents, used separately, were agent V, a neutralized proprietary resin, and agent X, a sodium lauryl sulfate type proprietary detergent.

PREPARATION, PROPORTIONS AND CURING

The batches were mixed in rotary tilt-drum 3- and 6-cu ft mixers operated at 75 percent capacity. Cement, water and air-entraining agent were mixed for 30 seconds; the aggregate was then added and mixed for 3 minutes, after which mixing was stopped for 2 minutes, and then resumed for a final 2-minute period. Agent V was introduced as a 2.5 percent solution; agent X was dissolved in water just prior to use with mix A, and introduced in the dry granular form in mix C. Water was added, when necessary, during the final two-minute mixing period to produce the required slump and air content. Several of the first batches mixed were found to have air contents well outside the selected ranges, 23 to 27 percent for mix A and 23 to 29 percent for mix C. These batches were discarded. With experience, control of the proportion of entrained air was improved, not by varying the quantity of air-entraining agent, but by careful adjustment of the quantity of mixing water during the final two-minute mixing period.

For each of the cement factor-admixture combinations (mixes AV, AX, CV, and CX) eight batches were made in the 6-cu ft mixer; one wallette and two cylinders were made from each

TABLE 1—TYPE OF SPECIMEN AND CURING AND TESTING SCHEDULE

Type of specimen	Days in molds	Days moist	Days dry*	Property tested for	Number of specimens tested	Age at test, days
6 x 12-in. cylinders	3	4	21	Compressive strength	72	28
6 x 12-in. cylinders	3	4	21	Dynamic elastic properties	56	28
6 x 12-in. cylinders	3	4	21	Stress-strain data	26	28
6 x 30 x 30-in. wallettes	6	1	21	Compressive strength	16	28
6 x 30 x 30-in. wallettes	6	1	21	Modulus of rupture†	16	28
6 x 6 x 12-in. pullouts	6	1	21	Bond stress-slip data and bond strength	15	28
2 x 2 x 12-in. prisms	3	4‡	180	Drying shrinkage	32	7 to 180
2 x 2 x 12-in. prisms	3	4‡	60	Freezing and thawing resistance	12	67 to 180
1 x 8 x 8-in. plates	3	4‡	21	Thermal conductivity	32	28 +§
1 x 8 x 8-in. plates	3	4‡	40	Water capillarity	16	47 +

*Drying occurred at 70 F and uncontrolled humidity except for shrinkage specimens which were dried at 70 F and 50 percent relative humidity.
†Pieces of wallettes broken in transverse test were used in capillarity, absorption and saturation coefficient determinations; these pieces were stored at room (uncontrolled) temperature and humidity for several weeks before testing.
‡Under water; other specimens were partially immersed and covered with damp burlap.
§Oven dried for 48 hours at 220 F prior to testing.

batch. One pull-out was made from each odd-numbered batch, and two prisms and two plates were made from each even-numbered batch. Additional batches were made in the 3-cu ft mixer, from which cylinders, prisms and plates were cast. Cylinders were rodded in the usual manner, and pull-outs, cast horizontally, were also rodded. The other types of specimens were not rodded but were spaded along the sides of the forms. Wallettes and plates were cast vertically (on edge), and concrete placed in several layers. All exposed surfaces were finished immediately after placing and then covered.

The proportions of the concretes, including entrained air content, and the densities (Table 2) were calculated from the weights of the 6 x 12-in. cylinders when removed from the molds at 3 days. The molds were watertight and covered, and it was assumed that the proportions

TABLE 2—PROPORTIONS, DENSITY, SLUMP AND WORKABILITY

Mix*	Number of batches	Air content, percent‡	Amount of admixture by weight of cement, percent	Cement factor, bags per cu yd	Water-cement ratio by weight	Air-cement ratio†	Aggregate-cement ratio†	Specific weight, lb per cu ft	Slump, in.	Workability
AV-I	5	23.3 ± 0.3	0.088	3.31	0.56	4.04	10.45	117.7	4.8	Poor to fair
AV-II	1	24.8	0.090	3.36	0.55	4.42	10.45	115.4	5.0	Good
AV-III	2	25.9 ± 0.1	0.090	3.21	0.54	4.63	10.45	114.2	4.5	Fair to good
AV-IV	2	27.6 ± 0.4	0.090	3.14	0.54	5.05	10.45	111.4	4.3	Very good
AX-I	3	21.8 ± 0.5	0.130	3.40	0.53	3.74	10.45	120.5	5.7	Poor to fair
AX-II	3	23.2 ± 0.4	0.130	3.35	0.53	3.99	10.45	118.4	4.3	Poor to fair
AX-III	2	24.5 ± 0.2	0.130	3.29	0.53	4.27	10.45	116.3	4.7	Poor to fair
AX-IV	3	27.4 ± 0.5	0.130	3.17	0.53	4.95	10.45	112.0	5.5	Very good
CV-I	3	22.5 ± 0.5	0.280	5.82	0.45	2.25	5.15	115.5	4.8	Very good
CV-II	2	25.9 ± 0.1	0.280	5.59	0.44	2.67	5.15	110.7	5.3	Very good to excellent
CV-III	3	27.5 ± 0.5	0.280	5.48	0.45	2.89	5.15	108.2	5.8	Excellent
CX-I	1	23.3	0.390	5.85	0.43	2.26	5.15	115.0	5.0	Excellent
CX-II	3	24.7 ± 0.4	0.390	5.73	0.43	2.46	5.15	113.1	6.2	Very good to excellent
CX-III	3	25.8 ± 0.1	0.390	5.66	0.43	2.61	5.15	111.8	6.0	Excellent
CX-IV	3	28.2 ± 0.5	0.390	5.51	0.41	2.92	5.15	107.8	6.5	Excellent
CX-V	1	29.7	0.390	5.36	0.43	3.18	5.15	106.0	7.0	Excellent

*The 3.3-bag mix is identified by the letter A and the 5.6-bag mix by the Letter C; V refers to the proprietary resin admixture, and X to a sodium lauryl sulfate type detergent admixture.
†By absolute volumes.
‡Batches are so grouped that variation in air content in each group is no greater than ± 0.5 percent; air contents were calculated on the basis of specific weights determined for 6 x 12-in. cylinders at 3 days.

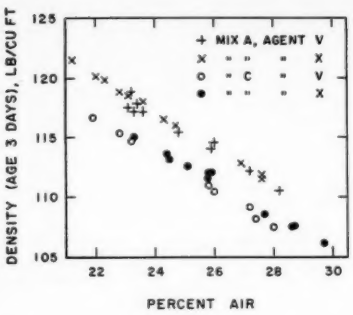


TABLE 3—AIR CONTENTS—PERCENT

Mix	No. 1, 0 Min.	No. 2, 10 Min.	Mean	No. 3, 3 days
AV	25.5	23.8	24.7	24.3
AX	26.4	22.9	24.6	24.1
CV	26.6	25.4	26.0	25.3
CX	26.4	24.8	25.6	26.1

Fig. 1—Density versus air content, calculated from weight of concretes at 3 days. (Mix A, 3.3, and Mix C, 5.6 bags cement per cu yd)

determined for the cylinders were the same as for the plastic mixes used in casting the specimens. The data in Table 2 are grouped in such a way that the variation in air contents among the batches represented in each group does not exceed ± 0.5 percent.

Table 3 summarizes the percentages of air calculated from density determinations made immediately following mixing (0 min.), after casting the specimens (10 min.), and at 3 days. Differences in determinations 1 and 2 indicate some instability of entrained air in each case. However, the means of determinations 1 and 2 made upon the plastic concretes agree well with determinations based upon the 3-day weights. Relationships between density and air content (Fig. 1) show that, for the same air content, the richer mix C weighed about 3 or 4 lb less per cu ft than mix A.

The variations in air content shown in Fig. 1 were obtained without varying, except in one minor case, the proportion of air-entraining agent used in each mix. Although some variation in air content was intended so that the influence of air content on the physical properties might be studied, the air content could not be predetermined as closely as desired.

Slump and workability characteristics are listed in Table 2. Workability was evaluated according to the relative ease of rodding or spading during fabrication of the specimens. Slump appeared to be a poor measure of workability. The sample of mix A concrete shown in Fig. 2A illustrates a deficiency in cement paste, despite an air content of 23 percent, while a second sample of mix A concrete, Fig. 2B, having an air content of 26 percent, was much more cohesive despite a lower slump. The first sample was "poor" and the second sample "good" in workability. The sample of mix C concrete, Fig. 2C, illustrates the generally excellent cohesiveness and workability of the richer mix; the air content of this sample was 27 percent. Examination of Table 2 indicates that the workability varied directly with air content.

It may also be noted in Table 2 that the water-cement ratios were relatively low—as much as 35 percent lower than for sand-gravel nonaerated concretes of comparable cement factors.

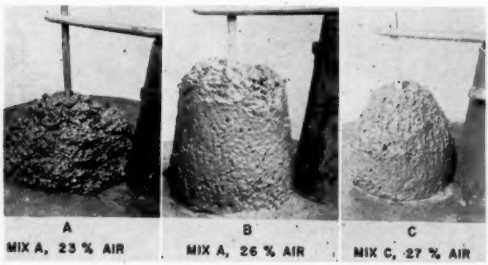


Fig. 2—Slump samples of "air-gravel" concretes

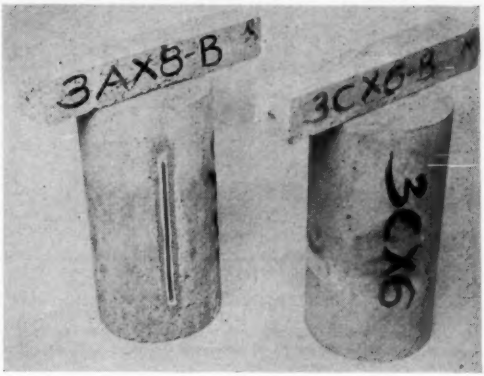


Fig. 3—Cylinders (6 x 12-in.) and prisms (2 x 2 x 12-in.), made from 3.3 bag mix (left), and 5.6 bag mix (right). A resistance strain gage is bonded to cylinder at left.

The curing and testing schedules for the various specimens are given in Table 1. None of the specimens was moist cured beyond 7 days. Prisms tested for freezing and thawing resistance were first subjected to drying for 60 days at 70 F and 50 percent relative humidity. The plates subjected to the capillarity test had previously been tested for thermal conductivity in an oven-dry condition. Determinations of absorption and saturation coefficient were made upon pieces of wallettes previously broken in the transverse tests.

RESULTS AND DISCUSSION

Cylinders and prisms (Fig. 3) and a pull-out (Fig. 4) had a fairly smooth cast surface texture, not unlike that of dense sand-gravel concretes. The texture of broken wallette surfaces is shown in Fig. 5.

Fig. 4—Pull-out specimen (6 x 6 x 12-in.) with dial gages for measuring slip of reinforcing bar at free and loaded ends.

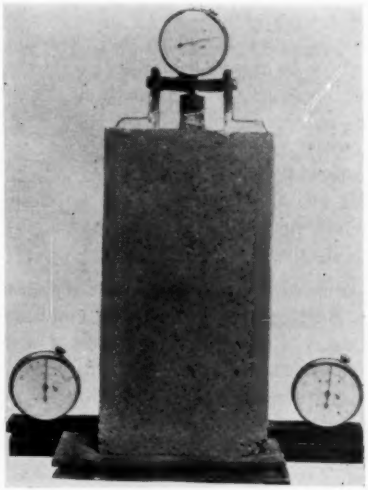
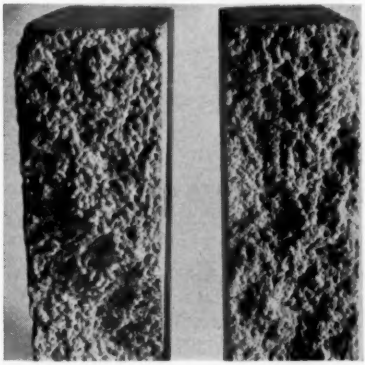


Fig. 5—Broken surfaces of 6 x 30 x 30-in. wallettes, 3.3 bag mix (left) and 5.6 bag mix (right)



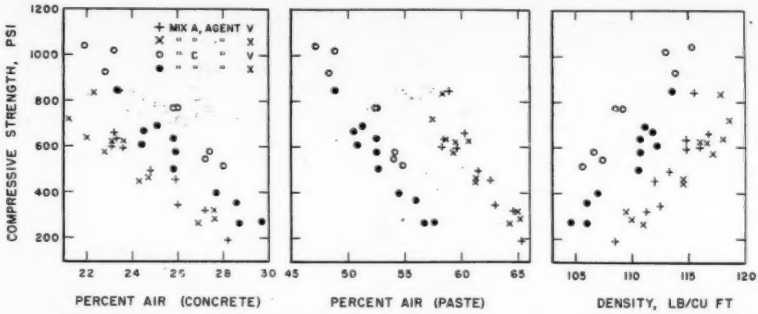


Fig. 6—Compressive strength of 6 x 12-in. cylinders versus air content of concrete (left), air content of cement paste constituent of concrete (center) and density of concrete (right). Strengths and densities are for 28 days.

Compressive strength of cylinders

Compressive strengths of 6 x 12-in. cylinders, determined at 28 days, are plotted in Fig. 6 against air content of the concrete, calculated air content of the cement paste in the concrete, and 28-day (room dry) density of the concrete. Each point represents the mean strength for two cylinders from a single batch. Fig. 6 indicates an extreme dependence of compressive strength upon air content and density; for each mix the compressive strength was lowered approximately 100 psi for each 1 percent increase in air content. It is also seen in Fig. 6 (left) that for the same air content, air-entraining agent V produced appreciably higher strengths than agent X when used in mix C.

In preparing Fig. 6 (center), the air content was calculated as a constituent of the cement paste only. The data for the two mixes differing in cement content fall into two well-defined groups. By extrapolating the data for mix A to lower air contents (or by extrapolating the data for mix C to higher air contents), it appears that for the same percentage of air in the cement paste components of the concretes, mix A would have a considerably—300 to 400 psi—higher compressive strength than mix C. The water-cement ratios of the leaner mix A, however, were almost one-fourth higher than those for mix C (Table 2). The apparently greater strength of mix A at a given paste air content may be related to the closer spacing of aggregate particles in that mix.

Compressive and transverse strength of wallettes

Wallettes were tested in compression in the position in which they were cast. In the transverse tests of wallettes, center loading was employed with the ends simply supported on a 27-in. span; these tests were made in such a way that the breaks would be perpendicular to the layers in which the specimens were cast.

Compressive and transverse strengths of wallettes are plotted in Fig. 7 against compressive strengths of 6 x 12-in. cylinders. Wallettes were generally

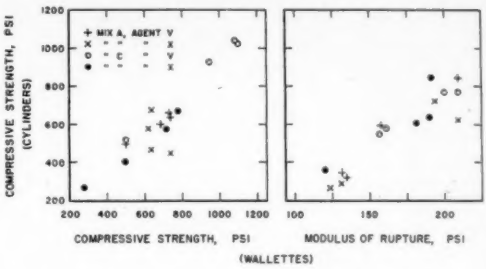


Fig. 7—Compressive strength of 6 x 12-in. cylinders versus compressive strength (left), and transverse strength (right) of 6 x 30 x 30-in. wallettes.

higher in compressive strength than cylinders from corresponding batches, particularly in the case of mixes containing agent X. The ratio of moduli of rupture of wallettes to compressive strengths of cylinders made from corresponding batches varied from about 0.25, for the stronger concretes, to about 0.40 for the weaker concretes. These ratios are considerably higher than those obtained for dense sand-gravel concrete and appear to be characteristic of relatively low-strength concretes.

Bond strength and slip characteristics of pull-outs

Fig. 4 shows the method of seating the 6 x 6 x 12-in. pull-out specimens upon a rubber-belting cushion, and the 0.0001-in. micrometer dial gages used for measuring the slip at the free (top) and loaded (bottom) ends of the specimen. A $\frac{7}{8}$-in. round, deformed reinforcing bar meeting the requirements of ASTM Standard A305-49 was used.

Bond-slip data are presented in Fig. 8, and bond strengths are plotted against compressive strengths of 6 x 12-in. cylinders in Fig. 9. The bond-slip characteristics for these concretes indicate that the maximum observed slip values for the free and loaded ends are considerably smaller than values obtained in similar tests of dense concrete.[6] Ratios of bond to compressive strength obtained for these concretes were approximately 0.25 to 0.5 and are considerably higher than corresponding ratios obtained for dense concrete; the higher ratios, as in the case of moduli of rupture, are apparently characteristic of concretes of relatively low strength. The slip data in Fig. 8 indicate an apparently greater brittleness than is shown by dense concretes.

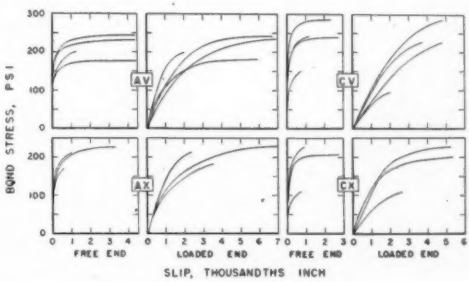


Fig. 8—Bond stress versus slip at free and loaded ends of 6 x 6 x 12-in. pull-out specimens. (Mix A, 3.3, and Mix C, 5.6 bags cement per cu yd; V and X, air-entraining agents)

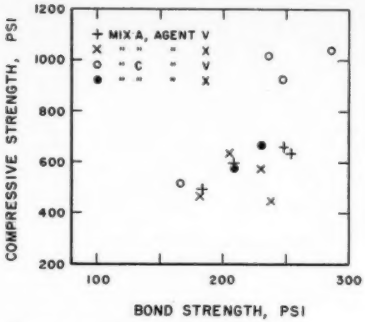


Fig. 9—Compressive strength of 6 x 12-in. cylinders versus bond strength of 6 x 6 x 12-in. pull-outs.

Dynamic and static elastic properties

Dynamic tests were made upon 6 x 12-in. cylinders at 28 days, prior to compressive strength tests. The dynamic modulus of elasticity for shear (modulus of rigidity, dynamic G), was calculated from the fundamental frequency of torsional vibration and the dynamic Young's modulus (dynamic E) was calculated from the fundamental frequency of longitudinal vibration of each specimen. In Fig. 10 the values for dynamic G and E are plotted against compressive strengths of cylinders. Poisson's ratio, μ, was calculated from values for dynamic G and E. The mean values were 0.17 for mix A, and 0.25 for mix C; no relationship between Poisson's ratio and other properties was evident.

Strain measurements were made on one 6 x 12-in. cylinder for each of four AV, six AX, eight CV, and eight CX batches. Two electric resistance strain gages were bonded to opposite sides of each cylinder (one gage is shown in Fig. 3). With the gages connected in series the indicated strain, representing the average of the strains measured by both gages, was measured by an a-c bridge type strain indicator. A secant modulus of elasticity was calculated for a strain equal to 0.0005 in. per in. The secant modulus is plotted against compressive strength in Fig. 10 (right).

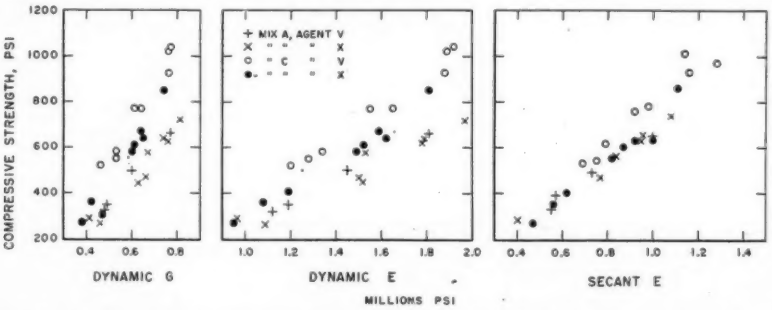


Fig. 10—Compressive strength versus dynamic modulus of rigidity (left), dynamic modulus of elasticity (center), and secant modulus of elasticity (right) for 6 x 12-in. cylinders. Secant modulus was calculated for a strain of 0.0005 in. per in.

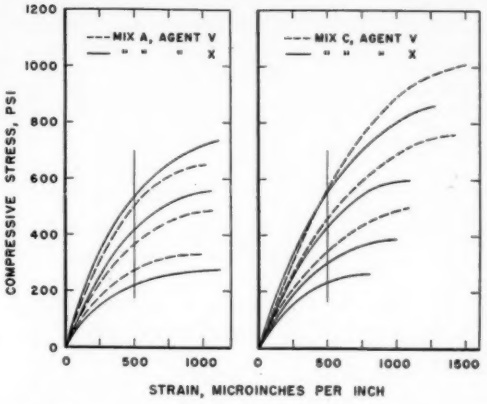


Fig. 11—Typical stress-strain curves for 6 x 12-in. cylinders. Secant modulus was calculated from such curves for a strain of 0.0005 in. per in. (note vertical line).

The mean ratios of secant moduli to dynamic Young's moduli were approximately 0.5 and 0.6 for mixes A and C respectively, appreciably lower than those obtained for dense sand-gravel concretes similarly cured and tested.

Typical stress-strain curves (Fig. 11) indicate that these concretes were apparently more brittle than conventional dense concrete; the maximum observed strains for the 3.3 bag mix A were about 0.001 in. per in., and for the 5.6 bag mix C, ranged from about 0.0010 to 0.0015 in. per in. Nonaerated sand-gravel concretes have generally shown maximum observed strains ranging from 0.002 to 0.003. In sand-gravel concretes 25 percent or more of the total strain usually occurs during the last increment of loading prior to failure.

Capillarity

Capillarity test specimens were pieces, approximately 6 x 10 x 15 in., from wallettes previously tested for transverse strength, and 1 x 8 x 8-in. plates previously tested for thermal conductivity. All specimens were immersed to a depth of about ¼ in. in water (Fig. 12). The tests were conducted at 70 ± 2 F and 82 ± 2 percent relative humidity. Capillary rise values are shown for each mix in Table 4; each value is a mean for the rise for four specimens from different batches of concrete. Nonaerated sand-gravel concrete plates showed about 4 in. capillary rise after 7 days. The wallette pieces had been "room-dried" under uncontrolled conditions for 5 to 10 weeks prior to test; the faces of the plates had been ground to remove the cast surface, oven dried

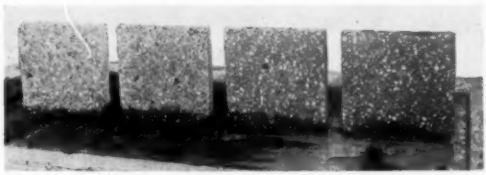


Fig. 12—Water capillarity test of 1 x 8 x 8-in. plates. These plates had previously been tested for thermal conductivity.

TABLE 4—CAPILLARY RISE, ABSORPTION AND SATURATION COEFFICIENT

Mix designation	Air content, percent*	Capillary rise, in.				Absorption, percent by volume§		Saturation coefficient**
		Wallettes†		Plates‡				
		1 hour	7 days	1 hour	7 days	24 hour,70 F	5 hour boil	
AV	25.1	0.5	0.9	0.7	0.9	7.9	28.8	0.28
AX	24.0	0.4	1.6	0.6	0.8	9.2	29.0	0.32
CV	26.6	0.5	0.6	0.7	0.8	7.9	31.1	0.26
CX	25.5	0.7	1.2	0.9	1.1	8.9	31.8	0.28

*Mean for four specimens, one from each of four batches.
†Portion of wallettes, 6 x 10 x 15-in.
‡1 x 8 x 8-in. plate with ground faces, previously tested for thermal conductivity.
§Portions of wallettes, 4 x 6 x 10-in.
**Ratio of 24-hour, 70 F to 5-hour boiling absorption.

prior to the conductivity testing, and then stored under uncontrolled conditions for several weeks before capillarity tests.

Absorption

Water absorption was determined upon pieces approximately 4 x 6 x 10 in. from wallettes previously tested for transverse strength; these specimens were oven dried for 64 hours at 230 F and then allowed to cool to room temperature. They were then wholly immersed in water for 24 hours at 70 F, after which they were boiled for 5 hours and then allowed to cool 16 hours while still immersed. Specimens were weighed after oven-drying, cold soaking and boiling, and the percentage of absorption, by volume, was calculated in accordance with ASTM Standard C 67-44. The saturation coefficient was calculated as the ratio of the 24-hour absorption to the 5-hour boiling absorption for each specimen. The results are presented in summary form in Table 4. The values shown for absorption are somewhat lower than those obtained for nonaerated sand-gravel concretes of comparable cement contents.[1] The saturation coefficient values are considerably lower than those obtained by similar methods for dense concrete, for which values ranging from 60 to 95 percent have been reported (Fig. 13).[7] While the 5-hour boiling absorption cannot be considered synonymous with the porosity of the concrete, it may indicate a lower limit for porosity. Mean values for the 5-hour boiling absorption were appreciably higher than the air contents calculated from the specific weight at 3 days.

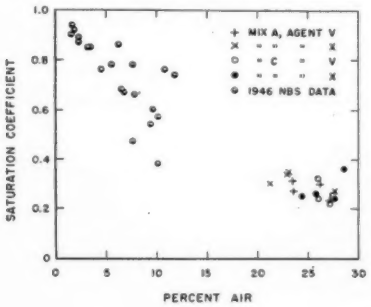


Fig. 13—Saturation coefficient versus air content. (Mix A, 3.3, and Mix C, 5.6 bags cement per cu yd; 1946 data are for sand-gravel concretes made with and without various air-entraining agents, and containing 5.5 bags cement per cu yd)

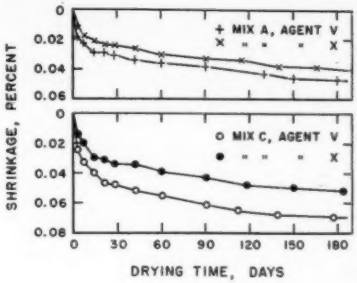


Fig. 14—Shrinkage versus drying time for 2 x 2 x 12-in. prisms. Each curve is a mean for four prisms from different batches.

Drying shrinkage

The 2 x 2 x 12-in. prisms, two of which are shown in Fig. 3, were removed from water at 7 days and allowed to dry at 70 ± 2 F and 50 ± 5 percent relative humidity. A horizontal comparator equipped with a micrometer screw and a 0.0001-in. dial gage for zero setting was used to measure length change. An initial reading was made immediately upon removal of each specimen from water, and thereafter shrinkage readings were made at convenient intervals. The mean shrinkage-drying time curves are shown in Fig. 14. Specimens from four batches are represented in each curve. The maximum shrinkage at 180 days shown by any specimen for each mix was:

Mix	*Air-entraining agent*	*Shrinkage, percent*
A	V	0.057
A	X	0.059
C	V	0.080
C	X	0.058

The mean percentages shown in Fig. 14 agreed to within a few thousandths with those obtained upon specimens made in preliminary stages of the study. Fig. 14 indicates that, in general, the richer mix showed greater shrinkage than the leaner mix when the same air-entraining agent was used, and that for the same cement factor mixes containing agent V showed greater shrinkage than those containing agent X. A nonaerated sand-gravel concrete made with the same brand and type of cement, White Marsh sand and pea gravel, and containing 6 bags of cement per cu yd, showed approximately 0.06 percent shrinkage after drying for 120 days.

Laboratory freezing and thawing tests

Freezing and thawing tests were made on 2 x 2 x 12-in. prisms previously dried at 70 ± 2 F and 50 ± 5 percent relative humidity for 60 days, and then wholly immersed in water for 72 hours. Specimens were subjected to one cycle of freezing and thawing a day, each cycle consisting of storage in air at 0 F for 16 hours and storage in water at 70 F for 8 hours. Determinations of dynamic *E*, discussed previously, were made prior to the beginning of the first cycle and at convenient intervals thereafter. The effects of cycles of freezing and thawing upon the specimens are indicated in Fig. 15, in which

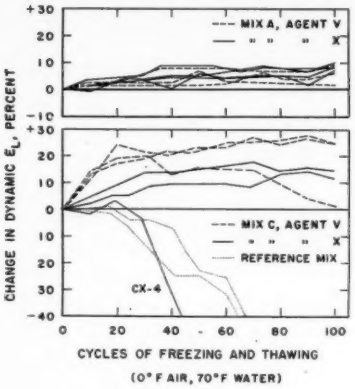


Fig. 15—Effect of laboratory freezing and thawing cycles upon the dynamic modulus of elasticity (longitudinal) of 2 x 2 x 12-in. prisms. (Mix A, 3.3, and Mix C, 5.6 bags cement per cu yd; reference mix, sand-gravel concrete containing 6 bags cement per cu yd and no entrained air)

changes in dynamic E are plotted against the number of cycles of freezing and thawing. While none of the seven specimens made from different batches of mixes AV and AX, appeared to be adversely affected by 100 cycles, one of the specimens of mix C concrete (specimen CX-4) "failed" in approximately 40 cycles.* Two specimens of nonaerated concrete made with the same materials as used for air-gravel concretes, and sand, but without an air-entraining agent, failed at about 60 cycles. The early failure of specimen CX-4 was probably related to the fact that concrete from this batch showed the highest absorption and saturation coefficient of all the air-gravel concretes. The absorption, 12.8 percent (by volume), was about one-third higher than for any other batch of mix C.

Thermal conductivity

Plates cast with dimensions 1⅛ x 8 x 8 in. were ground on both faces to remove the cast "skin" and to reduce the thickness to approximately 1 in. (Fig. 12). Specimens were oven dried at 220 F for 48 hours prior to testing in a guarded hot-plate thermal conductivity test apparatus. Apparatus and method are the same as used in an earlier study of lightweight aggregate concretes.[1] The thermal conductivity, or K values calculated from test data, and expressed in BTU per hour per sq ft of area per °F temperature difference per in. thickness, are plotted against density of concrete (oven-dry) in Fig. 16. Values are shown for two reference (sand-gravel) concrete specimens, as well as for air-gravel specimens. A K value often cited for dry dense concrete is 12, but values ranging from 7 to 16 have been reported by various observers. Values for the air-gravel specimens ranged from approximately 5 to 7, while values previously reported for nonaerated no-fines concretes ranged from approximately 8 to 16 for mixes of comparable density.[3]

*A 30 percent decrease in dynamic E is the criterion of "failure" used here.

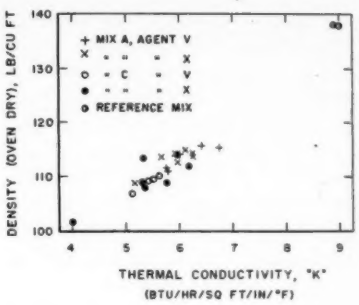


Fig. 16—Thermal conductivity, K values, versus density, for 1 x 8 x 8-in. plates. (Mix A, 3.3, and Mix C, 5.6 bags cement per cu yd; reference mix, sand-gravel concrete containing 6.0 bags cement per cu yd and no entrained air)

SUMMARY AND CONCLUSIONS

1. "Air-gravel" concretes, consisting of high-early strength portland cement, entrained air as fine aggregate and a siliceous pea gravel, were prepared using common types of rotary tilt-drum concrete mixers.

2. A proprietary neutralized resin and a detergent were used separately as air-entraining agents in concretes containing 3.3 and 5.6 bags of cement per cu yd, and having air contents of 22 to 30 percent. Control of air content was difficult; the quantity and method of addition of mixing water had a pronounced effect upon the quantity of air entrained, even when the amount of agent was held constant.

3. Compressive strength was quite sensitive to variations in air content, an increase of 1 percent in air content being generally accompanied by a decrease of 100 psi in compressive strength for air contents of 22 to 30 percent.

4. The indicated density of concretes having a compressive strength of 500 psi at 28 days was about 113 lb per cu ft for the 3.3 bag mix, and about 105 and 109 lb per cu ft, respectively, for 5.6 bag concretes made with resin and detergent air-entraining agents.

5. Ratio of modulus of rupture to compressive strength ranged from 0.25 to 0.40, for compressive strengths 800 to 300 psi, the lower ratios corresponding to the stronger concretes.

6. Ratio of pull-out bond strength to compressive strength ranged from 0.25 to 0.5, the lower ratios applying to the stronger concretes.

7. Dynamic Young's modulus ranged between 1 and 2 million psi; dynamically determined Poisson's ratios were about 0.18 for the 3.3 bag mix, and 0.25 for the 5.6 bag mix. The ratio of secant modulus for a strain of 0.0005 in. per in. to dynamic Young's modulus was about 0.5 for the 3.3 bag mix, and 0.6 for the 5.6 bag mix; the secant modulus appeared to be closely related to compressive strength and to dynamic moduli of elasticity and rigidity.

8. Relatively low values of maximum observed slip in the bond tests, and of maximum observed strain in the stress-strain determinations indicated that these concretes were more brittle than conventional dense sand-gravel concretes.

9. Maximum observed capillary rise of water in these concretes was relatively low—less than 2 in.—in relation to dense concrete.

10. Mean absorption (by volume) after 24-hour immersion in water was about 9 percent for all mixes. Saturation coefficients calculated as the ratio of 24-hour cold to 5-hour boiling absorptions ranged from 0.22 to 0.36, or about one-third values previously reported for nonaerated sand-gravel concrete.

11. Drying shrinkage of air-gravel concretes did not appear to differ significantly from values obtained for nonaerated sand-gravel concretes; the highest individual value observed for air-gravel concrete was 0.08 percent.

12. One hundred cycles of freezing and thawing produced one failure (a specimen of 5.6 bag concrete failed at 40 cycles) among 13 specimens of air-gravel concrete tested; two specimens of nonaerated sand-gravel concrete failed at 60 cycles.

13. Thermal conductivity values for air-gravel concretes were about 30 to 40 percent lower than for specimens of nonaerated sand-gravel concrete.

The most serious disadvantage of air-gravel concretes appears to be the difficulty of obtaining precise control of air content necessary to meet predetermined strength requirements. Additional work appears desirable with other types of air-entraining agents; foam stabilizers, accelerating agents and wetting agents might also be studied. The investigation of mixes having somewhat higher cement factors and air contents appears desirable. A promising field of study appears to be the design of aerated no-fines concretes containing various lightweight aggregates (see appendix). Reductions in density and water-cement ratio, and elimination of the often troublesome lightweight aggregate fines, appear to warrant such a study. Further studies in which dense aggregates are used might profitably be made to determine the effects of gradation and maximum size of aggregates.

ACKNOWLEDGMENT

The authors acknowledge the cooperation of the members of the staffs of the Structural Engineering and the Heating and Air-Conditioning Sections of the National Bureau of Standards, who assisted in preparing and testing the specimens.

Most of the work described in this paper was done with the financial support of the U. S. Bureau of Plant Industry, Soils and Agricultural Engineering.

REFERENCES

1. Kluge, R. W., Sparks, M. M. and Tuma, E. C., "Lightweight-Aggregate Concrete," ACI JOURNAL, May 1949, *Proc.* V. 45, p. 625.

2. Graf, O., *Gas Concrete, Foam Concrete, and Lightweight Lime Concrete*, Konrad Wittwer, Stuttgart, Germany, 1949.

3. Petersen, P. H., *National Bureau of Standards Building Materials and Structures Report* BMS 96, 1943.

4. Hummel, A., *Zement* (Germany), V. 25, 1936, p. 243.

5. Forlani, R., "No-Fines Concrete," ACI JOURNAL, Feb. 1950, *Proc.* V. 46, p. 477.

6. Clark, A. P., "Bond of Concrete Reinforcing Bars," ACI JOURNAL, Nov. 1949, *Proc.* V. 46, p. 161.

APPENDIX

Mixes containing California pumice (⅜- to ¾-in.), 5 bags of Type III cement per cu yd, air-entraining agent V, and no fine aggregate were made and cured as described in this paper. The density, air content and 28-day compressive strength for these mixes were:

Density, lb per cu ft		Air content, percent	Compressive strength, psi
3 days	28 days		
57	49	36.4	220
59	51	34.6	370
63	55	29.4	710
65	57	27.5	820

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

IN APPRECIATION

Those who look to CURRENT REVIEWS each month for information on new literature on concrete will join in thanking the reviewers for summarizing and evaluating a mass of published material for the reviews appearing in this section.

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IVAN M. VIEST, Urbana, Ill.
CHARLES C. ZOLLMAN, Kansas City, Mo.

Design of prismatic structures

A. J. ASHDOWN, Concrete Publications Ltd., London, 66 pp. $1.80

A design method for sloping reinforced concrete slabs for pitched roofs, the bottom of bunkers and other purposes is fully described with worked examples. Such structures are known as hipped-plate construction in the U. S. It is claimed that this type of construction has several advantages over thin curved shell roofs. These include cheaper forming, the fact that some parts can be precast and that the formulas and calculations are much simpler. The relaxation method of solving equations is clearly described.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 10, June 1951, *Proceedings* V. 47. Address 18263 W. McNichols Rd., Detroit 19, Mich. Copies of articles or books reviewed are **not** available through ACI. In most cases they can be obtained direct from the original publishers. Address, when available, will be furnished by ACI on request.

Aerated concrete—Part I

Digest No. 28, Mar. 1951, Building Research Station, Garston, Watford, Herts., England, 5 pp.

Brief description of methods of making foamed concrete and a discussion of the properties of several types.

Tension test on cylindrical or prismoidal specimens (L'Essai de traction simple sur eprouvettes prismatique ou cylinders)

MARCEL PROT, *Revue des Materiaux* (Paris), No. 426, Mar. 1951, pp. 69-76

Reviewed by PHILLIP L. MELVILLE

It is believed that a compression test does not give complete information on mortar and concrete mixes. Tension tests must be performed on duplicate specimens. Different size cylindrical or prismoidal molds having an h/d close to 3 are recommended. Special grips are used for tension tests.

Rectangular slabs subjected to flexure of loads normal to their plane (Placas rectangulares sometidas a flexion por cargas normales a su plano)

FRANCISCO GARCIA MONGE, *Instituto Tecnico de la Construccion y del Cemento* (Madrid), No. 75, 1949, 49 pp.

Reviewed by FRANCISCO J. CORDOVA

Various methods are presented for the analysis of rectangular slabs with supported, fixed and free edges. Several methods for the solution of Lagrange equation are exposed, together with the approximate solutions devised by several analysts. Working examples are shown and analyzed.

Experimental analysis of structures by means of bar models with one and two degrees of freedom (Analysis experimental de estructuras con modelos de barras con 1 y 2 grados de libertad)

JULIO RICALDONI, *Bulletin* No. 6, Instituto de Estatica, Facultad de Ingenieria (Montevideo, Uruguay), June 1949, 30 pp.

Reviewed by FRANCISCO J. CORDOVA

Professor Ricaldoni presents a study on the mechanical analysis of structures by means of models. Stress diagrams are analyzed mathematically by applying the stress and displacement methods. In both cases the structures are analyzed with two degrees of freedom.

Oblique flexure (Flexion esviada)

VICENTE CARINERA CASTELL, *Instituto Tecnico de la Construccion* (Madrid), No. 70, 28 pp.

Reviewed by FRANCISCO J. CORDOVA

The author discusses the different methods of analyzing structures or sections subjected to oblique flexure. The methods of Fischer, Hahn and Saeger, based on the elastic theory, and a new method based on a nonelastic theory are described with detailed mathematical formulas. The nonelastic method is based on Torroja's theory of reinforced concrete. Numerous stress diagrams and graphs are shown.

First general assembly of the International Building Information Council (Premiere Assemblee generale du Conseil International de Documentation du Batiment)

Cahier 104, Centre Scientifique et Techique du Batiment (Paris), 1950

Reviewed by PHILLIP L. MELVILLE

This publication contains the proceedings of the first assembly which met in Paris in October, 1950. Twenty-three countries, the United Nations, and a number of international organizations took part. The constitution and bylaws of the Council are followed by a list of committees. Committee activities are reported in detail.

Effect of blast-furnace slag cement on alkali-aggregate reaction in concrete

HERBERT P. COX, ROBERT B. COLEMAN, JR., and LOCKE WHITE, JR., *Pit and Quarry*, V. 43, No. 5, 1950, pp. 95-96

CERAMIC ABSTRACTS May, 1951

In work conducted at the Southern Research Institute, tests were made using blast-furnace slag cement and portland cement to determine the effect of each on the alkali-aggregate reaction. The results show that the slag cement nearly always inhibits alkali-aggregate expansion, and in no case is the magnitude of the expansion as great as would be expected if the alkali in the slag cement were actually aggravating the reaction.

Distribution of concrete stress in reinforced and prestressed concrete beams when tested to destruction by a pure bending moment

J. M. PRENTIS, *The Magazine of Concrete Research* (London), No. 5, Jan. 1951, pp. 73-77

AUTHOR'S SUMMARY

A method is given for utilizing experimental data derived from bending tests on reinforced and prestressed concrete beams to determine the stress-strain relationship for concrete. No initial assumption is made as to the shape of this curve, and the result obtained is unique for the given test data, so that

the factors which govern the validity of the final result are simply the basic assumptions of the method of analysis.

Fundamental principles for the analysis of plane-linear structures (Fundamentos para el calculo de estructuras lineales planas)

EDUARDO TORROJA, *Instituto Tecnico de la Construccion* (Madrid), No. 71, 1949, 38 pp.
Reviewed by FRANCISCO J. CORDOVA

Professor Torroja presents the fundamental principles of plane-linear structures. Various theorems are studied in detail accompanied by numerous figures and diagrams. From the theoretical point of view, this publication should be of great value to those who want to expand their knowledge of strength of materials.

Determination of stresses in simple and trussed beams (Determinacion de los esfuerzos en vigas simples y trianguladas)

EDUARDO TORROJO, *Instituto Tecnico de la Construccion* (Madrid), No. 74, 1949, 45 pp.
Reviewed by FRANCISCO J. CORDOVA

This is a continuation of the above article. The author presents the practical applications of the fundamental principles analyzed above. The article begins with the analysis of a simple beam and gradually expands to cover the more difficult calculations of trusses. The work is accompanied by extensive mathematical formulas, stress diagrams and schemes for calculations of stresses.

Highway engineering

L. J. RITTER, JR., and R. J. PAQUETTE, The Ronald Press Co., New York, 722 pp. $6.50

Factors in the development of a highway system are well-presented. Detailed information is given on administration, economics and finance, planning and geometric design of highways. In chronological order follow chapters on soil engineering, drainage and drainage structures, surveys and plans, contracts and supervision of construction and earthwork operations and equipment. The concrete engineer will find the material on portland cement concrete pavements disappointing since only 37 pages are devoted to their design and a scant 16 pages to their construction.

Reinforced concrete trusses (Cerchas de hormigon armado)

JOSE M. EYMAR, *Instituto de la Construccion y del Cemento* (Madrid), No. 76, 1949, 22 pp.
Reviewed by FRANCISCO J. CORDOVA

This publication was prepared to save designers and construction engineers the laborious calculations involved in the design of reinforced concrete trusses. A set of ten trusses are shown with spans varying between 8 and 26 m and a constant spacing of 5 m between trusses. The trusses are designed to support roofing material consisting of corrugated fibrous cement sheeting. The structural details of ten trusses are shown, two of which are analyzed in detail.

Stress diagrams, formulas and tables are included.

Control of granulated slags used in cement manufacture (Du controle des laitiers granules utilises en cimenterie)

LEON BLONDIAU, *Revue des Materiaux* (Paris), No. 424, Jan. 1951, pp. 6-9 and No. 425, Feb. 1951, pp. 42-6
Reviewed by PHILLIP L. MELVILLE

This study indicates the importance of the ratio CaO/SiO_2 in slags used in the manufacture of slag cements. It had been believed that granulation was the only requirement to cause hydraulicity of the resulting cement. It now appears that an excellent granulation is necessary but is not sufficient. Best results require a ratio CaO/SiO_2 between 1.45 and 1.54, a ratio SiO_2/Al_2O_3 between 1.8 and 1.9 and hydraulic potential (difference between the heats of hydration of a standard and a given slag cement) of 70 to 80 cal per g under given conditions. Heat of hydration was determined by ASTM method C-37.

Cements with reduced alkalinity (Ciments a basicite reduite)

FABIO FERRARI, *Revue des Materiaux* (Paris), No. 425, Feb. 1951, pp. 37-41
Reviewed by PHILLIP L. MELVILLE

The hydrolysis of C_3S results in a soluble, permeable calcium hydrate with low cementing power. The addition of a pozzolan would prevent the resulting disintegration, especially for underwater concrete. The resistance of pozzolanic cements to salt and sulfate waters is a direct function of the percent of C_3A and indicated by $Al_2O_3/Fe_2O_3 < 1$. It was found that pozzolanic cement mortars will harden in air as fast as ordinary mortars in wet curing. Best dur-

ability would be obtained by a reduced alkalinity cement having the following composition: SiO_2, 25 percent; Al_2O_3, 3 percent; Fe_2O_3, 5 percent; CaO, 64 percent; C_3S, 43 percent; C_4AF, 15 percent; silica ratio, 3.12; alumina ratio, 0.60 and lime 83. A list of 33 references is addended.

Combined use of pre-tensioning and post-tensioning methods

T. O. LAZARIDES, *The Magazine of Concrete Research* (London), No. 5, Jan. 1951, pp. 79-86

AUTHOR'S SUMMARY

The combined use of pre-tensioning and post-tensioning methods makes it possible to superimpose two systems of stresses in certain parts of a structure while other parts are stressed by one system only; the advantages of using pre-tensioning for the one component are that it introduces no anchors in the interior of the structure and facilitates the handling of component parts.

In continuous beams, individual pre-tensioned spans can be assembled by post-tensioning; the pre-tensioned component makes the spans safe in handling and also serves to counteract excessive stresses induced by the loads and by the post-tensioned component, thus making it possible to reduce the structural depth and keep the post-tensioning cables entirely straight. A slight sag in the post-tensioning cables in the end span can be advantageous and is usually permissible structurally.

Aqueous cementitious systems containing lime and alumina

HAROLD H. STEINOUR, *Bulletin* 34, Research and Development Division, Portland Cement Assn., Feb. 1951, 100 pp.

AUTHOR'S SUMMARY

This review covers in detail the literature on chemical equilibria that have significance relative to the composition of the aluminous products of portland cement hydration, and the nature of the setting process. Written primarily to provide appropriate background and perspective for future research, the review is not confined to the reactions of portland cement but is, in large part, a study of simpler systems of oxides: CaO–Al_2O_3–H_2O, CaO–Al_2O_3–$CaSO_4$–H_2O, and the 5-component systems involving alkalies. Thus it also covers aspects of the hydration of other cements, especially aluminous cement. It has been made unusually detailed and inclusive in the hope that it may materially reduce the time that new investigators need spend in similar study to gain a thorough comprehension of past work and its significance. The review shows that the problem of determining the aluminous products of portland cement hydration is far from being fully solved, and that many important questions remain to be answered relative to simpler systems. A detailed outline of contents, a factual summary, and subject and author indices are provided. 224 references.

Work of Oscar Niemeyer

Edited by STAMO PAPADAKI, Reinhold Publishing Co., New York, 220 pp. $8.50

Reviewed by NED H. ABRAMS

This volume is illustrative of possibly the most representative work of the present day in Brazil.

It should prove of interest—primarily to the architect, and secondarily to the engineer—since most of the presentation is photographic and descriptive, without an overabundance of technical detail or description. Many original structural forms are illustrated, but a great portion appear to be projected structures and one wonders if perhaps some are not somewhat in the position of trying to prove a point for the point's sake alone.

Imagination appears to be architect Niemeyer's forte and the volume is full of expressions of this capability of the architect. The influence of Le Corbusier is readily apparent in his work, and perhaps his most famous building, the Ministry of Education, Rio De Janerio, is a result of association with the famous French architect.

The book is more interesting from the standpoint of stimulating thought processes than from the viewpoint of developing a more intimate knowledge of the factors which provide Brazilian architecture and construction with the vital force it appears to possess.

Such a volume would be an interesting addition to one's library, but would not prove of much practical value other than to stimulate experimentation by those less conventional or timid souls who also want to prove new forms for themselves.

Some experiments on the load distribution in bond tests

R. J. WILKINS, *The Magazine of Concrete Research* (London), No. 5, Jan. 1951, pp. 65-72

AUTHOR'S SUMMARY

Tests are briefly described in which bond stress distribution has been obtained for tubes embedded in concrete and used as pull-out specimens. Electric resistance strain gages were inserted in straight lengths of steel tubing to form the measuring devices which can be used for direct calibration against load. These tubes were then embedded along the axis of cylinders of concrete to form pull-out specimens of the usual type. Readings of strain gages taken as the tube was being withdrawn from the concrete enabled the load distribution curves to be plotted. The variation of bond stress has been studied from information provided by load distribution curves, and values are given for average and maximum bond stress for various embedded lengths and types of tube surface. Results obtained from an American bar of the "Hi-bond" type with a patterned surface are included.

The experimental method has proved to be satisfactory in use and capable of providing sufficient data for plotting load distribution curves of reasonable accuracy. The transition of the load from the end at which it is applied to the free end of the tube has been observed and the method enables the mechanism of bond failure to be investigated. The most important influence appears to be the character of the contact suface, and it is considered that bond may be caused by (a) adhesion, (b) friction and (c) mechanical wedging. The relative importance of these factors is apparently controlled by the type of surface, but further work on this subject is in progress.

Some preliminary investigations in the use of bamboo for reinforcing concrete

S. R. MEHRA, H. L. UPPAL and L. R. CHADDA, *The Indian Concrete Journal*, V. XXV, Jan. 1951

HIGHWAY RESEARCH ABSTRACTS Apr. 1951

For some time attempts have been made to substitute for steel reinforcing in concrete some of the more common local materials such as wood, asbestos-cement and bamboo. Bamboo has shown promising laboratory results. Many species have high tensile strength; experiments in Germany showed that bamboo strips in tension have a modulus of elasticity of about 2,550,000 psi, approximately the same as that of concrete, and a resistance to tension of 28,250 psi. Similar tests at the Forest Research Institute, Deradun, India, on two of the commonly available species, *Bambusa Balcooa* and *Dendrocalamus strictus*, gave the following results:

1. Modulus of rupture in bending and tensile strength—14,000 psi
2. Modulus of elasticity—2.4×10^6 psi
3. Maximum crushing strength—6500 psi

The high tensile strength of bamboo recommends it as reinforcement. However, bamboo sometimes absorbs as high as 300 percent of its own weight in water and thereby increases in volume. When allowed to dry it loses water and shrinks back to its original volume. Such changes are undesirable in reinforcement. It has been suggested that this abnormal absorption can be considerably reduced if bamboo is at least three years old when cut. Absorption is further reduced if it is allowed to dry under mud. But even this absorption, about 50 percent, is unsafe. To further reduce absorption green bamboo was impregnated with various salt solutions. It is claimed that this treatment reduces volumetric changes considerably but in no case is it entirely eliminated.

The investigation covered waterproofing of bamboo, minimum cover of concrete required around bamboo reinforcement to prevent splitting of concrete and increasing bond of bamboo with concrete. Study of waterproofing bamboo has shown that a coat of rosin dissolved in alcohol with a subsequent painting of white lead reduces absorption to a point where volume change is almost negligible.

Inelastic behavior of engineering materials and structures

ALFRED H. FREUDENTHAL, John Wiley and Sons, Inc., New York, 1950, 587 pp. $7.50

Reviewed by DAN H. PLETTA

Structural and machine design depend on both the laws of mechanics and a knowledge of the behavior of materials. Rapid strides have been made in both fields in the past several decades, as evidenced by the expansion of analysis to the field of inelastic problems, and by the exploration of material response on a crystalline and an atomic

scale. Dr. Freudenthal's book correlates these advances in engineering science, developing the underlying principles of the behavior of materials so as to provide guides for further experimental research and thus a means for economizing on research effort and providing a rational basis for the interpretation of experimental data.

The book is based on an extended series of graduate lectures given originally at the University of Illinois and later at Columbia University. It deserves the attention of all designers, especially of those conversant only with the more elementary elastic theory of analysis, currently taught in most schools.

The introduction integrates the physical (structural) and engineering (phenomenological) approach of a material to the response of applied loads. The effects of time, temperature, etc., on physical response of materials and the vast discrepancy between the complex analysis of thermodynamic changes in gas turbines, or of aerodynamic loads, and the sometimes crude methods used to design the structural parts thereof, are mentioned as examples of engineering reality.

The text is then divided into three parts. The first of these deals with the structural aspect of mechanical behavior, including the structure of matter and theories of deformation. The second covers such subjects in the mechanics of inelastic deformation, as the equation of state, linear and plastic behavior, work hardening, creep, etc. The last part deals with the application of the mechanics of inelastic behavior, and discusses problems in torsion, bending, thick-walled cylinders, wire drawing, plastic flow in concrete, materials, testing, etc.

The book is well written and indexed. It is a truly professional contribution to engineering design and worthy of careful study.

PERIODICALS REGULARLY SCANNED FOR REVIEWS

The many requests for information concerning the source material of reviews appearing in the "Current Reviews" section indicate the desirability of publishing a list of those periodicals being regularly scanned for articles dealing with the field of concrete. Included also are the publishers' addresses for convenience should correspondence with them be desired.

Annales de L'Institut Technique du Batiment et des Travaux	28 Boulevard Raspail, Paris 7, France
Bauingenieur	Reichpietschufer, 20, Berlin, W35, Germany
Bautecknik, Die	Hohenzollerndam 169, Berlin-Wilmersdorf, Germany
Betong	Odengatan 41, Stockholm, Sweden
Beton og Jernbeton	Laboratoriet for Byningsteknik, Oster Volgade 10C, Copenhagen K, Denmark
Beton-und Stahlbetonbau	Hohenzollerndamm, 169, Berlin-Wilmersdorf, Germany
Beton-Teknik	Kalvebod Brygge 2, Copenhagen V, Denmark
Betonstein Zeitung	Frankfurter Strasse 5, Wiesbaden, Frankfurt Am Main, Germany
Bulletin, ASTM	1916 Race St., Philadelphia 3, Pa.
Bulletin, Institution of Civil Engineers in Ireland	35 Dawson St., Dublin, Ireland
Bulletin, University of Illinois	University of Illinois Engineering Experiment Station, 106 Engineering Hall, Urbana, Ill.
Bulletin Technique de la Suisse Romande	Case Postale Riponne, 21 Lausanne, Switzerland
Cahiers du Centre Scientifique et Technique du Batiment	33 Rue Jean-Goujon, Paris 8, France

Cement	Sarphatistraat 54, Amsterdam C, Netherlands
Cement and Lime Manufacture	14 Dartmouth St., London (Westminster S.W.1), England
Cement och Betong	Malmo, Sweden
Cemento, Il	9 Via Settembrini, Milan, Italy
Cemento Portland	Calle San Martin 1137, Buenos Aires, Argentina
Civil Engineer Corps Bulletin	Bureau of Yards and Docks, U.S.N., Washington 25, D. C.
Civil Engineering	33 W. 39th St., New York 18, N. Y.
Civil Engineering and Public Works Review	154 Fleet St., London E.C. 4, England
Commonwealth Engineer, The	349 Collins St., Melbourne, C. 1, Australia
Concrete	400 W. Madison St., Chicago 6, Ill.
Concrete and Constructional Engineering	14 Dartmouth St., Westminster S.W.1, England
Constructional Review	14 Spring St., Sidney, N.S.W., Australia
Crushed Stone Journal, The	1415 Elliot Place, N.W., Washington 7, D. C.
Current Road Problems	Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.
Engineering Experiment Station News	Ohio State University, Columbus, Ohio
Engineering News-Record	330 W. 42nd St., New York 18, N. Y.
Giornale del Genio Civile	Via delle Vite, 13, Rome, Italy
Hormigon Elastico	Lavalle 1171, 5°, Buenos Aires, Argentina
Handlingar (Proceedings)	Ingeniors Vetenskaps, Akademien, Stockholm, Sweden
Ingenioren	Copenhagen, Denmark
Journal of the American Ceramic Society	2525 N. High St., Columbus 2, Ohio
Journal of Research	National Bureau of Standards, Washington 25, D. C.
Magazine of Concrete Research	52 Grosvenor Gardens, London S.W.1, England
Osterreichische Bauzeitschrift	Mölkerbastei 5, Vienna 1, Austria
Polytechnisch Tydschrift	28, Balistraat, The Hague, Netherlands
Proceedings, ASCE	33 W. 39th St., New York 18, N. Y.
Public Roads	Bureau of Public Roads, U. S. Dept. of Commerce, Washington 25, D. C.
Reinforced Concrete Review, The	94-98 Petty France, London S.W.1, England
Revista de Obras Sanitarias de la Nation	Charcas 1840, Buenos Aires, Argentina
Revue des Materiaux	19 Rue Lafayette, Paris 9, France
Rock Products	309 W. Jackson Blvd., Chicago 6, Ill.
Structural Engineer, The	2 Bream Bldgs., London E.C.4, England
Technique Moderne-Construction, La	92 Rue Bonaparte, Paris 6, France
Teknisk Tidskrift	Kungsgatan 37, Stockholm C, Sweden
Teknisk-Ukeblad	Oslo, Norway
Travaux	6 Ave. Pierre 1st de Serbie, Paris 16, France
Zement-Kalk-Gips	Biebricher Strasse, 74, Wiesbaden, Germany